NEW ZEALAND

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NZS 4320 (2004) (English): Design of reinforced concrete masonry structures [By Authority of Compliance Document for Clause B1 Structure of the New Zealand Building Code]

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STANDARDS
NEW ZEALAND
PAEREWA AOTEAROA

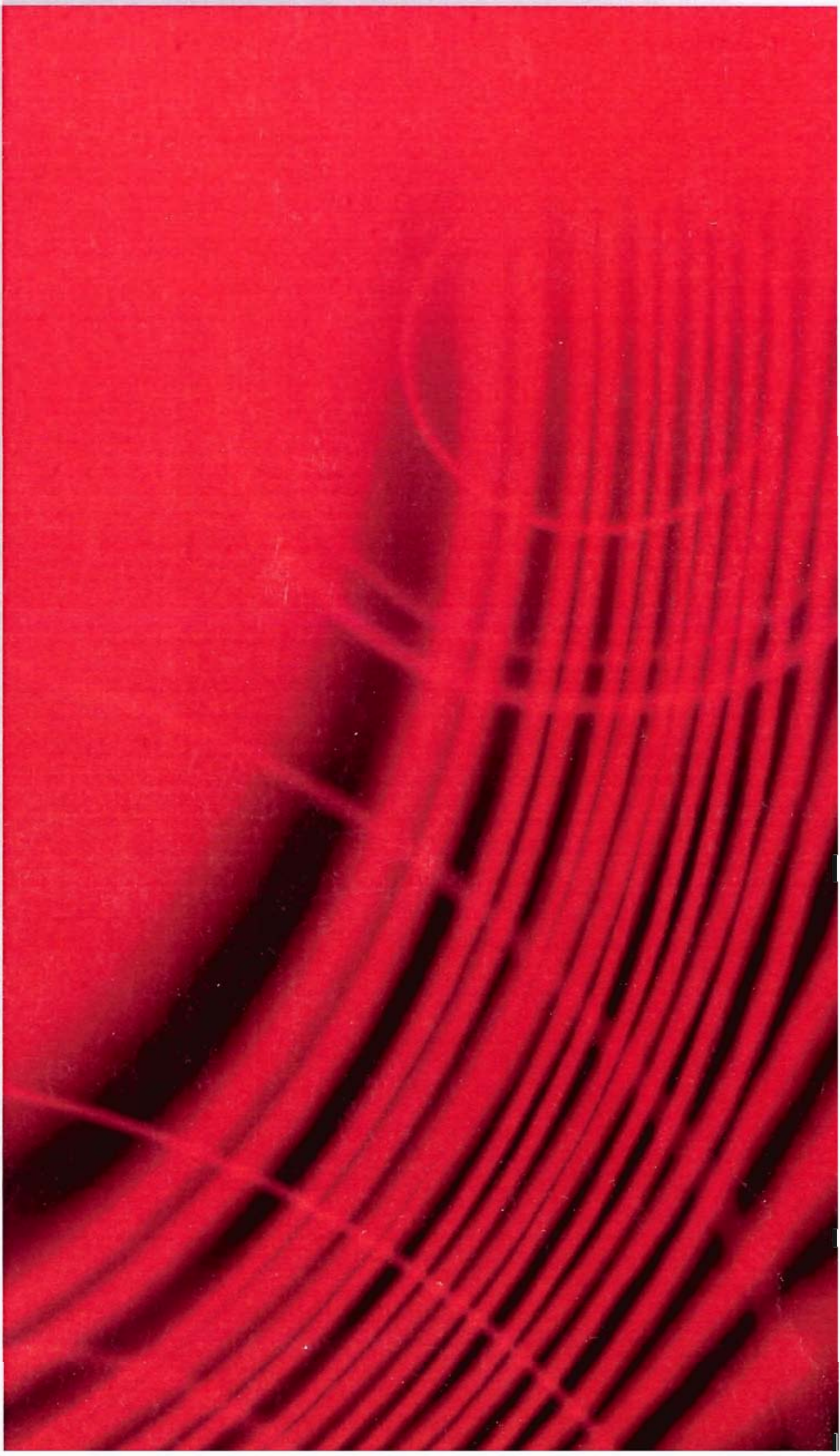
NZS 4230:2004
Incorporating Amendment No. 1

New Zealand Standard

Design of Reinforced Concrete Masonry Structures

Superseding NZS 4230:Parts 1 and 2:1990

NZS 4230:2004



COMMITTEE REPRESENTATION

This Standard was prepared under the supervision of the Design of Reinforced Concrete Masonry Structures Committee (P 4230) for the Standards Council established under the Standards Act 1988.

The Committee consisted of representatives of the following:

Building Industry Authority
Cement & Concrete Association of New Zealand
Co-opted Independent Chair
Institution of Professional Engineers New Zealand
Local Government New Zealand
New Zealand Concrete Masonry Association
University of Auckland

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In particular, Dr Jason Ingham and Mr Kok Choon (KC) Voon of the Department of Civil and Environmental Engineering at the University of Auckland provided valuable time and input into the development of the Standard. The doctoral research of Mr Voon contributed substantially to the updated approach to shear strength design.

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NZS 4230:2004

NEW ZEALAND STANDARD

DESIGN OF REINFORCED CONCRETE MASONRY STRUCTURES

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REFERENCED DOCUMENTS

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NZMP 9:1989	Fire properties of building materials and elements of structure	
NZS 1170.5:2004	Structural design actions:Part 5:Earthquake actions – New Zealand	
NZS 3101.1& 2:2006	Concrete structures Standard	Amd 1 Dec. '06
NZS 3109:1997	Concrete construction	
NZS 3112:- - - -	Methods of test for concrete	
Part 2:1986	Tests relating to the determination of strength of concrete	
NZS 3604:1999	Timber framed buildings	
		Amd 1 Dec. '06
NZS 4210:2001	Masonry construction – Materials and workmanship	
NZS 4229:1999	Concrete masonry buildings not requiring specific engineering design	

JOINT AUSTRALIAN/NEW ZEALAND STANDARDS

AS/NZS 1170:- - - -	Structural design actions	
Part 0:2002	General principles	
Part 1:2002	Structural design actions – Permanent, imposed and other actions	
Part 2:2002	Structural design actions – Wind actions	Amd 1 Dec. '06
Part 3:2003	Structural design actions – Snow and ice actions	
AS/NZS 1554:- - - -	Structural steel welding	
Part 3:2002	Welding of reinforcing steel	
AS/NZS 2699:- - - -	Built-in components for masonry construction	
Part 1:2000	Wall ties	
Part 2:2000	Connectors and accessories	
AS/NZS 4455:1997	Masonry units and segmental pavers	
AS/NZS 4456:2003	Masonry units, segmental pavers and flags – Methods of test	
AS/NZS 4671:2001	Steel reinforcing materials	
AS/NZS 4680:1999	Hot-dip galvanized (zinc) coatings on fabricated ferrous articles	

AUSTRALIAN STANDARDS

AS 1530:- - - -	Methods for fire tests on building materials, components and structures
Part 4:1997	Fire-resistance tests of elements of building construction
AS 3600:2001	Concrete structures
AS 3700:2001	Masonry structures

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BRITISH STANDARDS

BS 476:- - - -	Fire tests on building materials and structures
Part 20:1987	Method for determination of the fire resistance of elements of construction (general principles)
Part 21:1987	Methods for determination of the fire resistance of loadbearing elements of construction
Part 22:1987	Methods for determination of the fire resistance of non-loadbearing elements of construction
BS 5628:- - - -	Code of practice for use of masonry
Part 2:2000	Structural use of reinforced and prestressed masonry
BS EN 10088:	Stainless steels
Part 1:1995	List of stainless steels

AMERICAN STANDARDS

ACI 318-2002	Building code requirements for structural concrete
ASTM A370-03a	Standard test methods and definitions for mechanical testing of steel products
ASTM A416M-02	Standard specification for steel strand, uncoated seven-wire for prestressed concrete
ASTM A421M-02	Standard specification for uncoated stress-relieved steel wire for prestressed concrete
ASTM E111-97	Standard test method for Young's modulus, tangent modulus, and chord modulus

OTHER STANDARDS

ISO/CD 15835	Steel for the reinforcement of concrete – Mechanical splices for bars

OTHER PUBLICATIONS

Amd 1 Dec. '06 | Department of Building and Housing
The New Zealand Building Code (NZBC)

Building Research Association of New Zealand
Technical Recommendation No. 8: Method of fire engineering design of structural concrete beams and floor systems, 1991

LATEST REVISIONS

Users of this Standard should ensure that their copies of the above-mentioned New Zealand Standards and referenced overseas Standards are the latest revisions or include the latest amendments. Such amendments are listed in the annual Standards New Zealand Catalogue which is supplemented by lists contained in the monthly magazine *Standards Update* issued free of charge to committee and subscribing members of Standards New Zealand.

FOREWORD

Previous editions of this Standard have accommodated the generic composition of masonry. However this latest document recognizes the predominant use of reinforced concrete masonry for structural applications in New Zealand, and incorporates research findings specifically pertaining to the performance of reinforced and prestressed concrete masonry.

The content of this Standard, as with many other New Zealand Standards, is largely dictated by seismic considerations and is intended to provide satisfactory structural performance for concrete masonry structures during a major earthquake. There are minimum reinforcing requirements for different structural systems.

The basic design principles for reinforced concrete masonry are the same as for concrete and it is assumed that users of this Standard will have knowledge of reinforced concrete design.

This Standard allows for reinforced concrete masonry design within the limits of current knowledge, consistent with its known behaviour and possible seismic demands.

Although it is envisaged that a large amount of masonry will be designed using Observation Type B and requiring limited ductility, there is provision for ductile structures and higher grades of masonry where it is considered that additional detailing and supervision are warranted. This does permit a more economical structure to result from increased engineering input.

This document has been prepared on the basis of using the loading standard AS/NZS 1170.

Amd 1
Dec. '06

Review of Standard

Suggestions for improvement of this Standard will be welcomed. They should be sent to the Chief Executive, Standards New Zealand, Private Bag 2439, Wellington.

NOTES

NEW ZEALAND STANDARD

DESIGN OF REINFORCED CONCRETE MASONRY STRUCTURES

1 GENERAL

1.1 Scope

This New Zealand Standard specifies minimum requirements for the specific design of reinforced concrete masonry structures. It is also applicable to the design of parts of other buildings, which are constructed of reinforced concrete masonry and also for the use of prestressed concrete masonry in accordance with the limit state design method.

C1.1

This Standard applies to all designed reinforced concrete masonry construction, and to construction where concrete masonry forms only part of the construction. Where the concrete masonry is not part of either the vertical or horizontal load-carrying systems, design of such "mixed" construction is straightforward; where masonry and other materials act in conjunction in resisting loads care may be needed to determine behaviour, particularly in the post-elastic range.

This Standard applies to buildings which in the NZBC are defined as "any temporary or permanent movable or immovable structure (including any structure intended for occupation by people, animals, machinery, or chattels); and includes any mechanical, electrical, or other system, and any utility systems, attached to and forming part of the structure whose proper operation is necessary for compliance with the NZBC"; such as retaining walls. For other structures the designer should investigate the extent of applicability of the Standard to the particular structure, and seek the necessary approval.

General structural design and design loadings for buildings for compliance with the NZBC, are given in AS/NZS 1170.

Where construction methods are proposed which will differ from those of NZS 4210, an evaluation of those methods should be made before considering whether, or to what extent, this Standard should be used as a guide to design.

1.2 Interpretation

This Standard is intended for citation in Verification Method B1/VM1 of the Approved Documents for the New Zealand Building Code (NZBC) B1 "Structure". This Standard, however, is not a means of compliance with the NZBC by itself and must be used in conjunction with the loadings Standard AS/NZS 1170. Further, the use of the Standard is contingent on additional approvals being granted in respect of the engineering judgement made in the application of the Standard.

The terms "Normative" and "Informative" have been used in this Standard to define the application of the Appendix to which they apply. A "Normative" Appendix is an integral part of a Standard, whereas an "Informative" Appendix is only for information and guidance. Informative provisions do not form part of the mandatory requirements of the Standard.

1.2.1 *Interpretation of "shall" and "should"*
In this Standard the word "shall" identifies a mandatory requirement for compliance with the Standard. The word "should" refers to practices which are advised or recommended.

1.2.2 *Indication of commentary clauses*
Clauses prefixed by "*C*", printed in italic type and shaded, are intended as comments on the corresponding mandatory clauses. They are not to be taken as the only or complete interpretation of the corresponding clause nor should they be used for determining in any way the mandatory requirements of compliance within this Standard. The Standard can be complied with if the comment is ignored.

1.2.3 *Full titles in referenced document section*
The full titles of reference documents cited in this Standard are given in the list of Referenced Documents immediately preceding the Foreword.

1.2.4 *Non-specific requirements*
Where this Standard has non-specific requirements such as the words "suitable", "adequate", "acceptable" or other similar qualifiers like "as far as is reasonably practicable" then the method described shall be to the satisfaction of the territorial authority or building consent authority. Amd 1 Dec. '06

Also in this Standard, where reference is made to "the manufacturer's recommendations or instructions" or similar, these are outside the scope of this Standard and shall be to the satisfaction of the territorial authority or building consent authority. Amd 1 Dec. '06

Where this Standard requires special study then this is outside the scope of this Standard and shall be to the satisfaction of the territorial authority or building consent authority. Amd 1 Dec. '06

C1.2
The commentary not only explains the provisions of this Standard, but in certain cases it suggests approaches which satisfy the intent of this Standard. Where appropriate, a list of references is provided at the end of each section.

2 DEFINITIONS

2.1 General

For the purpose of this Standard the following definitions shall apply:

ACTION. Set of concentrated or distributed forces acting on a structure (direct action), or deformation imposed on a structure or constrained within it (indirect action). The term "load" is also often used to describe direct actions.

Amd 1
Dec. '06

BEAM. An element subjected primarily to loads producing flexure.

BOND

BOND – RUNNING OR STRETCHER. The unit set out when the units of each course overlap the units in the preceding course by between 25 % and 75 % of the length of the units.

BOND – STACK. The unit set out when the units of each course do not overlap the units of the preceding course by the amount specified for running or stretcher bond.

BUNDLES. Groups of parallel reinforcing bars bundled in contact, assumed to act as a unit, not more than two in any one bundle.

CAPACITY DESIGN. In the capacity design of earthquake resistant structures, elements of the primary lateral load resisting system are chosen and suitably designed and detailed for energy dissipation under severe deformations. All other structural elements are then provided with sufficient strength so that the chosen means of energy dissipation can be maintained.

CELL. A hole through or along a masonry unit in the plane of the wall where the least dimension of the hole exceeds one third of the width of the unit.

COLUMN. An element not longer than 790 mm having a minimum width of 240 mm subjected primarily to compressive axial load.

CONSTRUCTION OBSERVATION. Site inspections of typical sections of the work made from time to time with the object of deciding whether or not the work in place is constructed generally in accordance with the intent of the plans and specifications.

CONSTRUCTION SUPERVISION. More detailed examination of particular sections of the work than can be provided by the inspections made in the course of construction observation. This may require particular or continuous inspections as may be appropriate to decide whether or not the work in place is in accordance with the intent of the plans and specifications.

COVER. Distance from the nominal face of a reinforcing bar to the adjacent outside face of the masonry unit.

DESIGN ENGINEER. Any person who, on the basis of experience or qualifications, is competent to design structural elements of the structure under consideration to safely resist the design actions likely to be imposed on the structure.

DIMENSION. When used alone to describe masonry units means nominal dimension.

DUCTILITY. The ability of a structure to sustain its load carrying capacity and dissipate energy when it is subjected to cyclic inelastic displacements during an earthquake.

FACE-SHELLS. Those parts of a hollow unit which are connected by webs and which are normally laid in the plane of the wall.

Amd 1 Dec '06 FLUE. An enclosed continuous horizontal or vertical space in a masonry element formed by the cells of the masonry units which make up that member.

GALVANIZED. Hot-dip galvanized as specified in AS/NZS 4680.

GROSS AREA. The total cross-sectional area of a section through an element bounded by its external perimeter faces without reduction for the area of cells and re-entrant spaces.

GROUT. The material used to fill cells, grout spaces, or cavities in masonry.

GROUT SPACE. An enclosed continuous horizontal or vertical space in a masonry element formed by the cells of the masonry units which make up that component.

HOLLOW MASONRY UNIT. Unit with cores, intended to be laid with its cores vertical and with face-shell-bedded joints.

IN-FILL PANEL. A wall which is framed on four sides by beams and columns and contributes to shear resistance in the plane of the frame, but which is not designed to resist vertical loads other than its own weight.

IN-JOINT REINFORCEMENT. Steel wire/s system which has been hot-dip galvanized after fabrication and is to be embedded into a horizontal fresh mortar joint.

LOAD. See Action.

MASON. A person who, on the basis of qualification, training or experience, is competent to lay masonry in accordance with this Standard.

> REGISTERED MASON. A mason who is accepted for registration by the New Zealand Masonry Trades Registration Board and is the holder of a current registration certificate.

MASONRY. Any construction in units of concrete, laid to a bond, and joined together with mortar.

MASONRY UNIT. A preformed component intended for use in reinforced concrete masonry construction with cells laid in the vertical direction and with face-shell-bedded joints.

MORTAR. The cement/sand mix in which masonry units are bedded.

NET AREA. The gross cross-sectional area less the area of ungrouted cells and re-entrant spaces.

PARTITION. A non-loadbearing wall which is separated so as not to be part of the seismic resisting structure.

PILASTER. A vertical column formed integrally with a wall. It is an element subjected primarily to bending loads.

POTENTIAL PLASTIC HINGE REGION. Region in a component as defined in this Standard where significant rotations due to inelastic strains can develop under flexural actions.

PRIMARY ELEMENT. An element which is relied on as part of the seismic system.

REINFORCED MASONRY. Masonry which is reinforced to the minimum requirements of this Standard and grouted so that the two materials act together in resisting forces.

SECONDARY ELEMENT. An element which is not relied on as part of the seismic system but which may carry gravity or face loads or both in addition to its own weight, and meets the requirements of section 12.

SEISMIC SYSTEM. That portion of the structure which is considered to provide the earthquake resistance to the entire structure.

SPECIAL STUDY. A procedure for justifying departure from this Standard or for determining information not covered by this Standard. Special studies are outside the scope of this Standard.

STAINLESS STEEL. Describes austenitic stainless steel of grades 1.4301, 1.4436 or 1.4429 (formerly known as 302, 304, or 316) in the British Standard designation system as specified in BS EN 10088: Part 1).

STIRRUP. Reinforcement used to resist shear and torsion in a structural component, consisting of L-, U- or rectangular shapes and located perpendicular to, or at an angle to longitudinal reinforcement.

STRENGTH

STRENGTH, DESIGN. The nominal strength multiplied by the appropriate strength reduction factor.

STRENGTH, LOWER CHARACTERISTIC YIELD, (of steel). The value of yield strength below which not more than 5 % of production tests in each size falls.

STRENGTH, NOMINAL. The theoretical strength of a component section, calculated using the section dimensions as detailed and the theoretical characteristic material strengths as defined in this Standard.

STRENGTH, OVER. The overstrength of a section takes into account all possible factors that may contribute to strength, such as higher than specified strengths of steel and masonry, steel strain hardening, and additional steel placed for construction and which may not have been accounted for in calculations.

STRENGTH, REQUIRED. The strength of a component section required to resist combinations of actions for ultimate limit states as specified in AS/NZS 1170: Part 0.

STRENGTH, SPECIFIED COMPRESSIVE, (of masonry). A singular value of strength normally at age 28 days as determined in Appendix B unless stated otherwise, denoted by the symbol f'_m which classifies masonry as to its strength class for purposes of design and construction.

Amd 1 Dec '06

STRUCTURAL. A term used to denote an element or elements which are required to provide resistance to actions imposed on the building.

TIE REINFORCEMENT. Tie means reinforcement used in a confining or lateral restraint role usually in the shape of a hoop, square, rectangle, or cross tie or link, located perpendicular to, or at an angle to the longitudinal reinforcement. Ties may also be used to resist shear and torsion in a structural component.

WALL. A vertical element, which because of its position and shape contributes to the rigidity and strength of a structure.

C2.1

For consistent application of the Standard, it is necessary that terms be defined where they have particular meanings in the Standard. The definitions given are for use in the application of this Standard only and do not always correspond to ordinary usage. AS/NZS 1170 refers to the suite of Loading Standards comprising AS/NZS 1170.0, AS/NZS 1170.1, AS/NZS 1170.2, AS/NZS 1170.3 and NZS 1170.5.

Amd 1 Dec '06

3 LIMIT STATE DESIGN REQUIREMENTS AND MATERIAL PROPERTIES

3.1 Notation

c	Depth of neutral axis from the compression edge of the section, mm
d	Distance from extreme compression fibre to centroid of tension reinforcement, mm
E_m	Modulus of elasticity of masonry, MPa
E_s	Modulus of elasticity of reinforcement, MPa
f_y	Reinforcement yield strength, MPa
f'_m	Specified compressive strength of masonry, MPa
M^*_E	Design moment for component resulting from earthquake loading, specified in AS/NZS 1170, Nm
M^*_G	Design moment for component resulting from gravity loading, specified in AS/NZS 1170, Nm
M_n	Nominal flexural strength of section, Nm
M^*_Qu	Design moment for component resulting from live loading, specified in AS/NZS 1170, Nm
S^*	Ultimate limit state design actions, specified in AS/NZS 1170
S_n	Nominal strength of component for given design action
S_p	Structural performance factor
T_1	Fundamental period of the building
V^*_E	Design shear for component resulting from earthquake loading, specified in AS/NZS 1170, N
V^*_G	Design shear for component resulting from gravity loading, specified in AS/NZS 1170, N
V_n	Nominal shear strength of section, N
V^*_Qu	Design shear for component resulting from live loading, specified in AS/NZS 1170, N
ε_u	Ultimate compression strain of masonry
ϕ	Strength reduction factor
μ	Structural ductility factor, specified in AS/NZS 1170

C3.1

The following symbols which appear in this clause of the commentary are additional to those used in section 3.

ε_dc	Shrinkage coefficient of a masonry unit
ε_sh	Shrinkage coefficient of concrete masonry

3.2 Scope

Provisions of this section apply to the general design and construction of masonry structures. Requirements for different observation types of masonry, methods of design loading arrangements, and assumptions for analysis shall be as specified in this section. Detailed requirements for design shall be in accordance with subsequent sections of this Standard.

3.3 General principles and requirements for construction

3.3.1 *General*

3.3.1.1 *Requirements for masonry units*
All masonry units shall:

(a) Comply with AS/NZS 4455 and strength requirements of NZS 4210;

(b) Be tested in accordance with AS/NZS 4456;

(c) Be of such type and arrangement that will be conducive to complete filling of all the grouted cells.

3.3.1.2 *Construction requirements*
Construction shall conform with NZS 4210.

3.3.2 *Masonry types*

3.3.2.1 *Observation types*
Masonry shall be classified into observation types constructed and supervised in accordance with the requirements of table 3.1.

Table 3.1 – Observation types, admissible use and design compressive strengths

Amd 1
Dec. '06

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Dec. '06

Observation type	**Observation requirement**	**Admissible use**	**Maximum specified compressive strength of masonry** (MPa)
C	May be built without construction observation by a design engineer or a nominated representative thereof.	• Elastic and nominally ductile structures. • Face loaded walls designed for limited ductility.	4
B	Shall be inspected by a design engineer or by a nominated representative thereof, who may be a mason deemed to comply with the competency requirements of NZS 4210. Such inspection shall establish that the design is being interpreted correctly and that the work is being carried out generally as specified.	Elastic, nominally ductile, limited ductile or ductile structures.	12
A	In addition to the inspection required of Type B, Type A observation of masonry shall require construction supervision at all critical stages by a person approved by a design engineer, having appropriate knowledge/experience of correct masonry trade practices and reporting to a design engineer, such as to ensure that the standards of materials and workmanship applying on the job are of a consistently high quality commensurate with the achievement of superior strengths. Masonry shall be constructed using a mason deemed to comply with the competency requirements of NZS 4210.	Elastic, nominally ductile, limited ductile or ductile structures.	≥12 A higher design f'_m may be used if substantiated by testing in accordance with Appendix B.

C3.3.2.1
All masonry construction must comply with NZS 4210 Masonry construction – Materials and workmanship. *As such the tradesperson must work to the requirements of NZS 4210 which in turn requires an expression of trade competency.*

Type C Observation
This is primarily in use for masonry constructed within the requirements of NZS 3604.

Type B Observation
This is primarily the default method for most structural masonry including masonry constructed to the requirements of NZS 4229.

Type A Observation
This is primarily required when the designer wishes to use special masonry construction features that are not provided for in terms of construction requirements by NZS 4210. Typical examples would be the use of prestressing, confining plates and enhanced strength of materials which will require a project quality assurance testing programme. In Type A work, it would be expected that a registered structural mason would be required to carry out construction but under the independent observation of the designer or his/her representative.

This Standard refers to the use of a Registered Mason. Registered Structural Masons must have had their previous work independently assessed by Chartered Professional Engineer(s) or Professional Members of IPENZ, taken an examination on the content of NZS 4210 and possess a current copy of NZS 4210, before the independent NZ Masonry Trades Registration Board will grant registration. The Board maintains a list of Registered Structural Masons on the website www.mtrb.org.nz

3.3.3 *Identification of types of observation*
The design engineer shall identify on the drawings or in the specification the observation requirements of all the masonry elements. Where more than one type is used in one structure the parts to which the respective type apply shall be clearly identified on the drawings.

3.4 Material and strength properties

3.4.1 *Masonry strengths*
Amd 1 Dec '06 Observation type-dependent design compressive strengths for compression, f'_m, shall be in accordance with the values given in table 3.1.

C3.4.1
Table 3.1 gives the design strengths to be used for compression. The Type C observation value relates to a historical decision to arbitrarily limit the value to 4 MPa. This conservative value is primarily used in NZS 3604 where qualified masonry trades people were not necessarily being used. Values chosen for Type B Observation are based on the default minimum values in NZS 4210 of 12.5 MPa masonry units and 17.5 MPa for grout infill. Except where design is based on higher than observation Type A minima stresses, the design strength of masonry is not controlled by specifying a minimum prism strength, but rather by specifying minimum strengths and properties of component materials, and the way in which these component materials are built up into masonry elements and the quality of the workmanship used.

3.4.2 *Modulus of elasticity of masonry*
The modulus of elasticity, E_m, shall be taken as 15 000 MPa for all masonry structures.

3.4.3 *Ultimate compressive strain for unconfined masonry*
For unconfined masonry, the available ductility shall be based on an ultimate compression strain of $\varepsilon_u = 0.003$.

3.4.4 *Modulus of elasticity of reinforcement*
The modulus of elasticity, E_s, of non-prestressed steel reinforcement shall be taken as 200 000 MPa.

3.4.5 *Reinforcement strength*
Design shall be based on a lower characteristic yield strength for non-prestressed reinforcing steel, f_y, not in excess of 500 MPa.

3.4.6 *Modulus of elasticity of prestressing tendons*

The modulus of elasticity, E_s, of prestressing tendons shall be determined by tests to ASTM A370-03a Annex A7 to comply with ASTM E111-97.

3.4.7 *Strength reduction factors, ϕ*

Strength reduction factors, ϕ, shall be as follows:

Flexure with or without axial tension or compression	0.85
Axial tension	0.85
Bearing on masonry	0.65
Shear and shear friction	0.75
Design for fire exposure	1.0
Strut and tie models	0.75

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However, when the design moments, axial loads, or shear forces for a section are derived from overstrengths of adjacent components or sections, in accordance with capacity design principles, a strength reduction factor of $\phi = 1$ shall be adopted.

3.5 Limit state requirement for design

3.5.1 *General*

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The structure and its components shall be designed to satisfy the requirements of this Standard for stiffness, strength and ductility. The relevant combinations of actions specified for each of the serviceability and ultimate limit states in AS/NZS 1170 shall be used for design in accordance with this Standard.

3.5.2 *Serviceability limit state*

3.5.2.1 *General*

The structure and its components shall be designed for the serviceability limit state by limiting deflection, cracking and vibration in accordance with the relevant requirements of this Standard and to the serviceability requirements of AS/NZS 1170.

3.5.2.2 *Stiffness*

Components shall be designed to have adequate stiffness to limit deflections or any deformations which may adversely affect the serviceability of the structure.

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C3.5.2.2
For guidance on section properties to be used in seismic analysis refer to NZS 3101.

3.5.2.3 *Seismic actions*

Assessment of structural deflections for the serviceability limit state involving seismic forces shall make due allowance for anticipated levels of masonry cracking. For the serviceability limit state, deflections shall be calculated using a structural performance factor, S_p, of 0.7 in the determination of the applied actions.

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3.5.2.4 *Cracking*

Cracking of masonry at the serviceability limit state shall be limited so that the durability of the structure is not adversely affected having regard to the requirements of the particular structure.

C3.5.2.4
There is limited information specifically related to acceptable crack widths in reinforced concrete masonry and it is recommended that reference be made to the provisions of NZS 3101 for guidance on acceptable values.

n reinforced concrete masonry, the development of flexural cracking is generally dictated by the bond strength between the mortar and the masonry unit. Consequently, this parameter dictates the magnitude of the Modulus of Rupture (MOR). NZS 4210 specifies a minimum bond strength at 7 days of 200 kPa, but permits designers using NZS 4230 to specify a greater value. Furthermore, 28-day values will be greater than the 7-day strength.

The topic has received little research attention in New Zealand, but the US Masonry Standards Joint Committee[3.1] *has suggested values for the Modulus of Rupture. These values are based upon the behaviour of unreinforced masonry, with the conservative decision made by that committee that the adopted values should not be amplified for reinforced masonry.*

For out-of-plane loading, typical US Modulus of Rupture values expressed for mortar complying with NZS 4210 indicate that when using a running bond pattern, MOR values of 860 kPa and 1380 kPa may be taken for hollow and fully grouted masonry respectively. For stack bonded structural masonry, MOR values of 430 kPa and 1170 kPa respectively are specified. MOR values for partially filled masonry can be determined by interpolation between the corresponding values of hollow and fully grouted masonry, based on the percentage of grouting.

For in-plane loading, the recommended US value of the Modulus of Rupture is 1.7 MPa. For a specified compression strength of masonry of 12 MPa (see table 3.1) this corresponds to approximately $0.5\sqrt{f'_m}$ *which is suitably conservative when compared with the value specified in NZS 3101 for structural concrete.*

3.5.2.5 *Vibration*

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Appropriate measures shall be taken to evaluate and limit where necessary the effects of potential vibration from wind actions, machinery and vehicular or pedestrian traffic movements on the structure, occupants and contents.

3.5.2.6 *Shrinkage and shrinkage/thermal control joints*

Where it is deemed necessary to make specific provision for shrinkage and control joints in a concrete masonry structure in order to accommodate the effects of drying shrinkage and/or thermal movements, the design engineer shall provide appropriate details as to the nature and location of such control joints.

C3.5.2.6

All concrete masonry products will undergo shrinkage, which is dependant upon the composition of materials used in the manufacture. Where specific use of a shrinkage value is needed e.g. prestressed masonry, the designer is advised to contact potential suppliers of units to obtain characteristic shrinkage values for units to be supplied from the particular masonry manufacturing plant. Shrinkage tests should be conducted in compliance with AS/NZS 4456.

The US Masonry Standards Joint Committee[3.1] *has specified a shrinkage strain of masonry as* $\varepsilon_{sh} = 0.5\,\varepsilon_{dc}$ *as a practical evaluation of shrinkage in practice for* $\varepsilon_{dc} > 0.00065$*, or* $0.15\,\varepsilon_{dc}$ *for* $\varepsilon_{dc} \leq 0.00065$*. The characteristic* ε_{dc} *of most lightweight masonry units will be > 0.0008.*

Designers should be cognizant that during the grouting process, face-shells of fully grouted masonry regularly absorb significant quantities of water, such that any attempt to limit shrinkage prior to grouting through conditioning of the masonry units may have little effect on post-grout shrinkage behaviour.

A long-term shrinkage study[3.2] *using concrete masonry units comprised of lightweight pumice aggregate was conducted at the University of Auckland to quantify vertical shrinkage in order to establish prestress losses in prestressed masonry. Based upon that study it was recommended that for determining prestress losses when using vertically oriented prestress tendons, a shrinkage strain,* ε_{sh}*, of 0.0006 is appropriate for ungrouted masonry, and a post-grouting shrinkage strain of 0.0010 is appropriate for fully grouted masonry.*

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The placing of shrinkage and thermal control joints in walls is a matter of judgement by the designer with consideration being given to the type of construction, shape of walls (accounting for features such as openings) and the amount of reinforcement in the walls and exposure to the weather. When the reinforced concrete masonry contains only the minimum of 0.07 % of horizontal reinforcing, it is suggested that shrinkage control joints be used where weaknesses occur and at every 6 to 8 m and that thermal control joints be used every 30 to 50 m. It is recommended that where joints are to occur close to large openings the joint should be relocated to the side of that opening.

Research on reinforced hollow masonry suggests more specific guidelines on control joint placement, based on walls having a minimum horizontal reinforcement content of 0.03 %, as follows:

(a) At horizontal spacing not exceeding the lesser of 1.5 times the height of the wall or 7.6 m;

(b) At changes in wall height;

(c) At changes in wall thickness, such as at pipe and duct chases and pilasters;

(d) At (above) movement joints in foundations and floors;

(e) At (below) movement joints in roofs and floors that bear on a wall;

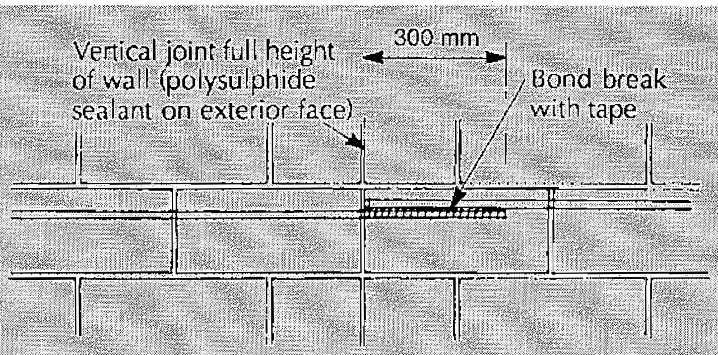
(f) Near one side of door or window openings less than 1.8 m wide or on both sides if the opening is wider (joints can be away from the opening if adequate tensile reinforcement is placed around the opening);

(g) Adjacent to corners of walls or intersections within a distance equal to half the control joint spacing.

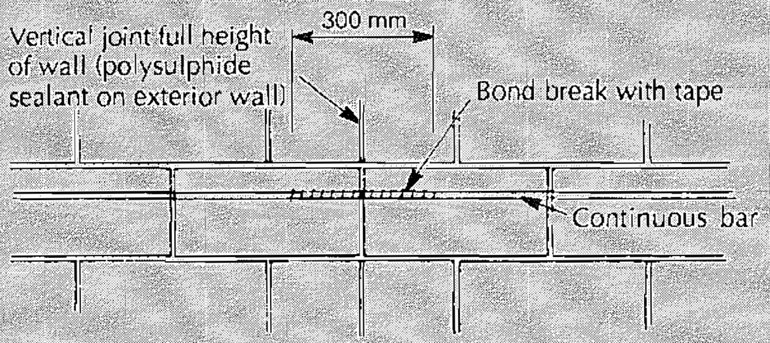
These recommendations are considered suitable for application to New Zealand reinforced concrete masonry.

Shrinkage control joints which have performed well in practice are shown in figure 3.1.

When control joints have to be located within a structural component it is recommended that they should be located close to the middle of the wall rather than near the outer edge, which will be subjected to inelastic strains. In these instances, however, the shear forces in the wall have to be carried across the vertical joint by shear friction.


Vertical joint full height of wall (polysulphide sealant on exterior face)
300 mm
Bond break with tape

(a) Control joint separating two walls


Vertical joint full height of wall (polysulphide sealant on exterior wall)
300 mm
Bond break with tape
Continuous bar

(b) Control joint within single wall

Figure 3.1 – Shrinkage control joints

3.5.2.7 *Durability*

Masonry shall be designed to comply with the provisions of section 4 of this Standard.

3.5.3 *Ultimate limit state requirements*

3.5.3.1 *Design for strength*

Structures and structural components shall be designed for strength as follows:

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(a) The ultimate limit state design action, S^*, shall be determined from the governing ultimate limit state combinations specified in AS/NZS 1170;

(b) The design strength of a component or cross-section at the ultimate limit state shall be taken as the nominal strength, S_n, for the relevant action calculated in accordance with the requirements and assumptions of this Standard, multiplied by the applicable strength reduction factor, ϕ, specified in 3.4.7;

(c) The component shall be proportioned so that the design strength is not less than the design action, in accordance with the following relationship:

$$\phi S_n \geq S^* \qquad \text{(Eq. 3-1)}$$

where S is replaced in Eq. 3-1 by the actions of moment, axial force, shear or torsion as appropriate.

3.5.3.2 *Design for stability*

For ultimate limit state load combinations not involving earthquake, the structure as a whole and its components shall be designed to prevent instability due to overturning, sliding or uplift in accordance with AS/NZS 1170.

3.5.3.3 *Design for fire resistance*

The fire resistance of concrete masonry construction shall comply with section 5 of this Standard.

3.5.3.4 *Seismic actions – ultimate limit state*

The ultimate limit state inter-storey deflection determined in accordance with NZS 1170.5 shall not exceed 2.5 % of the corresponding storey height.

3.5.4 *Use of test data*

Amd 1 Dec '06 Use of test data to satisfy the requirements of limit state design may be based on Appendix B of AS/NZS 1170.0. Use of data in this way is not part of the verification method for the Building Code.

3.6 General principles, assumptions, and requirements for analysis and design

3.6.1 *Assumptions and methods of analysis*

3.6.1.1 *Design action effects*

Amd 1 Dec '06 All components of frames or continuous construction shall be designed for the maximum design action effects as determined by the theory of elastic analysis, except as modified in accordance with 3.6.1.5. The redistribution of moments permitted in 3.6.1.5 shall not be applied to the approximate moments of 3.6.1.2.

3.6.1.2 *Design of continuous beams using approximate actions*

Approximate moments and shears may be used in design of continuous beams in accordance with the provisions of NZS 3101.

3.6.1.3 *Stiffness*

The following general requirements apply to stiffness:

(a) Calculation of the flexural, shear, and torsional stiffness of structural components shall be based on recognized engineering principles. Assumptions shall be consistent throughout analysis;

(b) The effect of stiff panel zones at the intersection of deep components and haunches shall be considered both in determining bending moments and in design of components.

3.6.1.4 *Moment of inertia for T-beams and flanged shear walls*

In computing the effective moment of inertia of cracked sections, the effective width of the overhanging parts of flanged components shall be one half of that given in 3.6.1.7.

3.6.1.5 *Moment redistribution*

Redistribution of the design moments obtained by elastic analysis may be carried out for non-prestressed concrete masonry components subjected to flexure in accordance with all the following provisions:

Amd 1 Dec '06 (a) Equilibrium between the internal forces and the external loads must be maintained under each appropriate combination of factored actions;

(b) The dependable strength after redistribution, provided at any section of a component, shall not be less than 70 % of the moment for that section obtained from an elastic moments envelope covering all appropriate combinations of unfactored actions; Amd 1 Dec '06

Amd 1 Dec '06 (c) The elastic moment at any section in a component due to a particular combination of design actions shall not be reduced by more than 15 % of the numerically largest moment given anywhere by the elastic moments envelope for that component, covering all combinations of design actions;

(d) The neutral axis depth, c, of a section resisting a reduced moment due to moment redistribution shall not be greater than:

$$c = 0.25\,d \qquad \text{(Eq. 3-2)};$$

(e) The consequences of redistribution assumed at the ultimate limit state shall be assessed for the serviceability limit state.

3.6.1.6 *Span lengths*

For the purpose of calculating moments, shears, deflections, or stiffnesses the following span lengths shall be used:

(a) Span length of components not built integrally with supports shall be considered to be the clear span plus depth of component but need not exceed the distance between centres of supports;

(b) In analysis of frames or continuous construction for determination of moments, the span length shall be taken as the distance centre-to-centre of supports;

(c) For beams built integrally with supports, moments at faces of support shall be used for design.

3.6.1.7 *Effective widths for T-beam and flanged shear walls*

In T-beam construction, and in flanged shear walls, the flange and web shall be built integrally unless shown by a special study to be otherwise effectively bonded together, and the following shall apply:

(a) The effective width of a flange resisting action due to flexure shall not exceed one quarter of the span length of the beam or one third of the height of the shear wall, whichever is appropriate, and the effective overhanging flange width on each side of the web shall not exceed:

  (i) Eight times the flange thickness; nor

  (ii) Half the clear distance to the next web.

(b) For beams or shear walls with a flange on one side only, the effective overhanging slab width considered in flexural resistance shall not exceed:

  (i) One twelfth the span length of the beam, or one ninth of the height of the shear wall, whichever is appropriate; nor

  (ii) Six times the slab thickness; nor

  (iii) Half the clear distance to the next web.

(c) Isolated beams, in which the T-shape is used to provide a flange for additional compression area, shall have a flange thickness not less than one half the width of the web and an effective flange width not more than four times the width of the web. In such beams transverse reinforcement placed perpendicularly to the beam shall be provided so as to:

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  (i) Carry the design action on the overhanging slab width assumed to act as a cantilever;

  (ii) Act as shear reinforcement when necessary to ensure flange action;

  (iii) Be placed not further apart than five times the slab thickness, nor 450 mm.

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3.6.1.8 *Structurally irregular buildings*

Structurally irregular buildings are as defined in NZS 1170.5.

3.7 Principles and requirements for components designed for seismic loading

3.7.1 *Assumptions and methods of analysis and design*

3.7.1.1 *Design philosophies*

To provide appropriate resistance for the combination of gravity and seismic loads specified by AS/NZS 1170, design methods and design parameters shall be used which are applicable to the design philosophy as summarized in table 3.2.

Table 3.2 – Design parameters for various design philosophies

Design philosophy	Seismic performance	Structural ductility factor, μ	Structural performance factor, S_p	Required grouting	Method of design
Elastic structures	Potential to form soft stories or brittle failure modes	1.0	1.0	Solid filled or partially filled acceptable	Design exempt from additional seismic requirements. Design shall be in accordance with 3.7.2
Nominally ductile structures	Design to avoid soft stories or brittle failure modes	1.25	0.9	Solid filled or partially filled acceptable	Design exempt from additional seismic requirements. Design shall be in accordance with 3.7.2
Limited ductile structures	Limited dissipation of energy by flexural yielding in specified locations	2	0.7	Solid filled in potential plastic hinge regions. Other regions may be solid or partially filled	Design procedures as outlined in 3.7.3 or capacity design as defined in section 2
Ductile structures	Dissipation of energy by ductile flexural yielding in specified locations	4	0.7	Solid filled	Design procedures as outlined in 3.7.4 including capacity design as defined in section 2

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NOTE – The S_p is for the ultimate limit state condition and for the serviceability condition. See 3.5.2.3.

C3.7.1.1

For limited ductile and ductile structures, the design must be based on the assumption that under combined gravity and earthquake loading, lateral deformations will be sufficiently large to develop a complete potential plastic hinge mechanism in the structure. The lateral earthquake loading must be considered in both directions and hence this will usually involve reversed potential plastic hinges. The design philosophy of table 3.2 is consistent with that of NZS 1170.5. The limit of $\mu = 4$ for ductile structures in table 3.2 has been set to ensure that the serviceability limit state design actions do not exceed the ultimate limit state design actions.

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3.7.1.2 *Interaction of structural and non-structural elements*

The interaction of primary and secondary structural elements, which due to seismic displacements may affect structural response or the performance of non-structural elements, shall be considered in the design. Requirements for design of secondary structural elements are given in section 12.

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Consequences of failure of elements that are not part of the intended primary system for resisting seismic actions shall be considered.

3.7.1.3 *Design of floors and roofs*

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Floor and roof systems in buildings shall be designed to act as horizontal structural elements, where required, to transfer seismic actions to frames or structural walls.

3.7.1.4 *Use of structural ductility factor in equations*

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In the derivation of the lateral seismic loading to be considered with the appropriate gravity load, the structural ductility factor, μ specified in table 3.2 shall be used. The same structural ductility factor, μ, shall be substituted in all relevant equations of the additional seismic requirements of this Standard.

3.7.1.5 *Design for concurrency*

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The effects of concurrency in two-way horizontal force resisting systems in elastic and nominally ductile structures shall be accommodated in accordance with 5.3.1.2 of NZS 1170.5. Ductile structures designed using capacity design principles in accordance with 3.7.4 shall be designed for concurrency in accordance with 2.6.5.8 of NZS 3101.

3.7.1.6 *Strength reduction factors*

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In determining the design strengths for components designed for the maximum effects of static loads determined by elastic analysis, or for effects derived from dynamic analysis as permitted by AS/NZS 1170, the strength reduction factors specified in 3.4.7 shall be used.

3.7.1.7 *Potential plastic hinges assumed to form in ductile and limited ductile structures*

Structures classified as limited ductile or ductile frames composed of beams and columns with or without shear walls, and also cantilever or coupled shear walls, shall be assumed to be forced into lateral deformations sufficient to create potential plastic hinges by actions of a severe earthquake.

3.7.1.8 *Effects of cracking on stiffness*

For the purpose of estimating periods of vibration and structural deformations, to comply with requirements of AS/NZS 1170, allowances shall be made for the effects of:

(a) Cracking on the stiffness of various structural components;

(b) Stiffness or deformations of shear walls and other deep components when considering shear distortions, and distortions of anchorages and foundations.

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C3.7.1.8

For guidance on section properties to be used in seismic analysis refer to NZS 3101.

3.7.1.9 *Structures outside those covered in this Standard*

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Structural systems and design methods other than those covered in this Standard shall be the subject of a special study as defined in Appendix A of AS/NZS 1170.0.

3.7.2 *Elastic and nominally ductile responding structure*

Structures which are expected to respond elastically or in a nominally ductile manner to large earthquake motions, in accordance with table 3.2, are exempt from the additional seismic requirements of all relevant sections of this Standard, provided that the earthquake design load used is that specified for these types of structures by NZS 1170.5. For such structures, strength design procedures in accordance with the general principles and requirements of the relevant sections of this Standard shall be used.

3.7.3 *Limited ductile structures*

3.7.3.1 *General*

In limited ductile structures, the system as a whole or the primary lateral load resisting components are not considered to be capable of sustaining the inelastic displacements that are expected in ductile structures, without significant loss of strength or reduction in energy dissipating capacity. Therefore the design of such structures is in accordance with table 3.2 provided that:

(a) Structural ductility factor, μ, as defined in table 3.2 shall be used to derive the total design earthquake load to the requirements of AS/NZS 1170;

(b) Appropriate detailing of potential plastic hinge regions, in accordance with limitations imposed for this structure, shall be adopted to ensure that the reduced ductility demands can be met;

(c) The design forces and detailing requirement shall conform with either 3.7.3.2 or 3.7.3.3.

3.7.3.2 *Design using a capacity design approach*
For a capacity design approach:

(a) No height restrictions apply to structures designed to this design method;

(b) Design forces shall be calculated using a capacity design approach as specified in 3.7.4.

3.7.3.3 *Design using a simplified capacity design approach*
When using a simplified capacity design approach the following shall be considered:

Amd 1 Dec '06 (a) Structures designed using this design philosophy shall be wall structures not exceeding three storeys or four storeys with a light roof as defined in NZS 4229. The maximum storey height shall be 3.6m.

(b) Design for concurrent earthquake effects from loading in two principal directions is not required for structural components designed to meet the requirements of this clause;

Amd 1 Dec '06 (c) Design flexural strength outside the designated plastic hinge region shall be such that:

$$\phi M_n \geq M_G^* + M_{Qu}^* + 1.5 M_E^* \quad \text{.......... (Eq. 3-3)}$$

Amd 1 Dec '06 (d) Design shear strength shall have a suitable margin over the required flexural strengths, such that:

$$\phi V_n \geq V_G^* + V_{Qu}^* + 2 V_E^* \quad \text{.......... (Eq. 3-4)}$$

(e) Design shall be in accordance with the additional principles and requirements for structures designed using a limited ductile design philosophy provided in relevant sections of this Standard;

Amd 1 Dec '06 (f) The structure shall be classified as regular using the definitions provided in NZS 1170.5.

3.7.4 *Ductile structures*

3.7.4.1 *Capacity design*
Wherever the requirements of a capacity design procedure apply, the maximum component actions to be expected during large structural deformations shall be based on the flexural overstrength of the potential plastic hinges. However no component is required to be designed to resist forces greater than those corresponding to the use of a structural ductility factor of 1.0 and structural performance factor, S_p, of 0.7.

C3.7.4.1
When components are designed on the basis of load combinations that include earthquake actions, consideration is given to the maximum likely actions that could ever develop on the section from beam flexural overstrengths at potential plastic hinges or from an adjacent component or components. For this extreme loading case, when significant inelastic deformations and damage in various parts of the structure are to be expected, additional reserve strength is not considered to be necessary. Therefore the nominal strength of a component should be equal to or greater than the load demand derived from a capacity design procedure. Consequently in capacity design, strength reduction factors need not be used, or alternatively, where relevant it should be assumed that $\phi = 1$.

Because most masonry will be unconfined, the available ductility will be limited by the unconfined compression strain of 0.003. The clauses in this Standard imposing limits to depth of compression zone of flexural components as a means for insuring adequate ductility, have been based on the assumption of no redistribution of design bending moments. Moment redistribution will impose additional ductility demand on hinges whose design moment has been reduced from the value predicted on the basis of elastic analysis, and hence should not be used unless calculations show that the required additional ductility is available.

3.7.4.2 *Calculation of flexural overstrength*

The flexural overstrength in a potential plastic hinge shall be calculated allowing for strain hardening and material variation of the reinforcement provided. Unless a special study is conducted, the overstrength of reinforcement shall be:

(a) 1.25 f_y for grade 300, class E reinforcement;

(b) 1.4 f_y for grade 500, class E reinforcement.

3.7.4.3 *Dynamic magnification*

In ductile structures where the lateral earthquake load is resisted by a system of moment resisting frames or cantilever or coupled walls, the appropriate structural ductility factor, μ, specified in table 3.2, shall be used, and where applicable, allowance for the dynamic magnification of shear forces shall be made in accordance with NZS 3101.

3.7.4.4 *Design for maximum shear force*

Capacity design procedures shall be used to ensure that the nominal shear strength of masonry elements is in excess of the shear force acting when flexural overstrength is reached.

3.7.4.5 *Ductile coupled shear walls*

When cantilever walls are interconnected in the same plane at intervals by substantial ductile beams, part of the seismic energy to be dissipated shall be assigned to the coupling system. Design procedures shall be used to ensure that the energy dissipation in the coupling system can be maintained at its flexural overstrength, without exceeding the ultimate compression strain of the concrete masonry.

C3.7.4.5

General design requirements, including dynamic shear magnifications, are discussed in NZS 3101.

3.7.4.6 *Ductile moment resisting frames*

General design principles for ductile masonry moment resisting frames shall comply with the requirements of NZS 3101 including allowance for P-delta effects where required.

3.7.4.7 *Ductile hybrid structures*

Whenever a combination of different ductile structural systems is used, rational analysis, taking into account the relative stiffness and location of such elements, shall be employed to allocate the seismic resistance to each element. Attention shall be given to the likely energy dissipation capacity of each element, and the ensuing local damage in relation to the ductility demand on the element when the desired ductility for the building as a whole is attained.

3.7.5 *Foundations*

General design principles for concrete masonry foundations shall comply with the requirements of NZS 3101.

3.7.6 *Structures incorporating mechanical energy dissipating devices*

The design principles associated with incorporation of mechanical energy dissipation devices shall be the subject of a special study.

3.7.7 *Secondary structural elements*

Secondary elements shall be designed to the requirements of section 12.

REFERENCES

3.1 MSJC (2002). "Building Code Requirements for Masonry Structures" and "Specification for Masonry Structures", ACI 530-02/ASCE 5-02/TMS 402-02, Masonry Standards Joint Committee, USA.

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3.2 Laursen, P.T., Wight, G., Ingham, J.M., "Assessing Creep and Shrinkage Losses in Post-tensioned Concrete Masonry", ACI Material Journal, Nov-Dec 2006.

4 DESIGN FOR DURABILITY

4.1 Scope

This section may be used in conjunction with NZS 3101 to provide a means of compliance with Clause B2 of the NZBC.

4.2 Classification of sites into sea spray zones or corrosion zones

4.2.1 *Sea spray and corrosion zones*

Building sites shall be classified as being in sea spray zones or corrosion zones 1, 2, 3 or 4, depending on the severity of exposure to wind-driven sea salt or to geothermal gases.

Sea spray zones and corrosion zones 1, 2, 3 and 4 are shown in figure 4.1.

4.2.1.1 *Location of sea spray zone*

The sea spray zone referred to in table 4.1 is defined as within 500 m of the sea including harbours, or 100 m from tidal estuaries and sheltered inlets, as well as areas shown unshaded on figure 4.1. The sea spray zone also includes all offshore islands including Waiheke, Great Barrier Island, Stewart Island and the Chatham Islands.

4.2.1.2 *Location of geothermal hot spots*

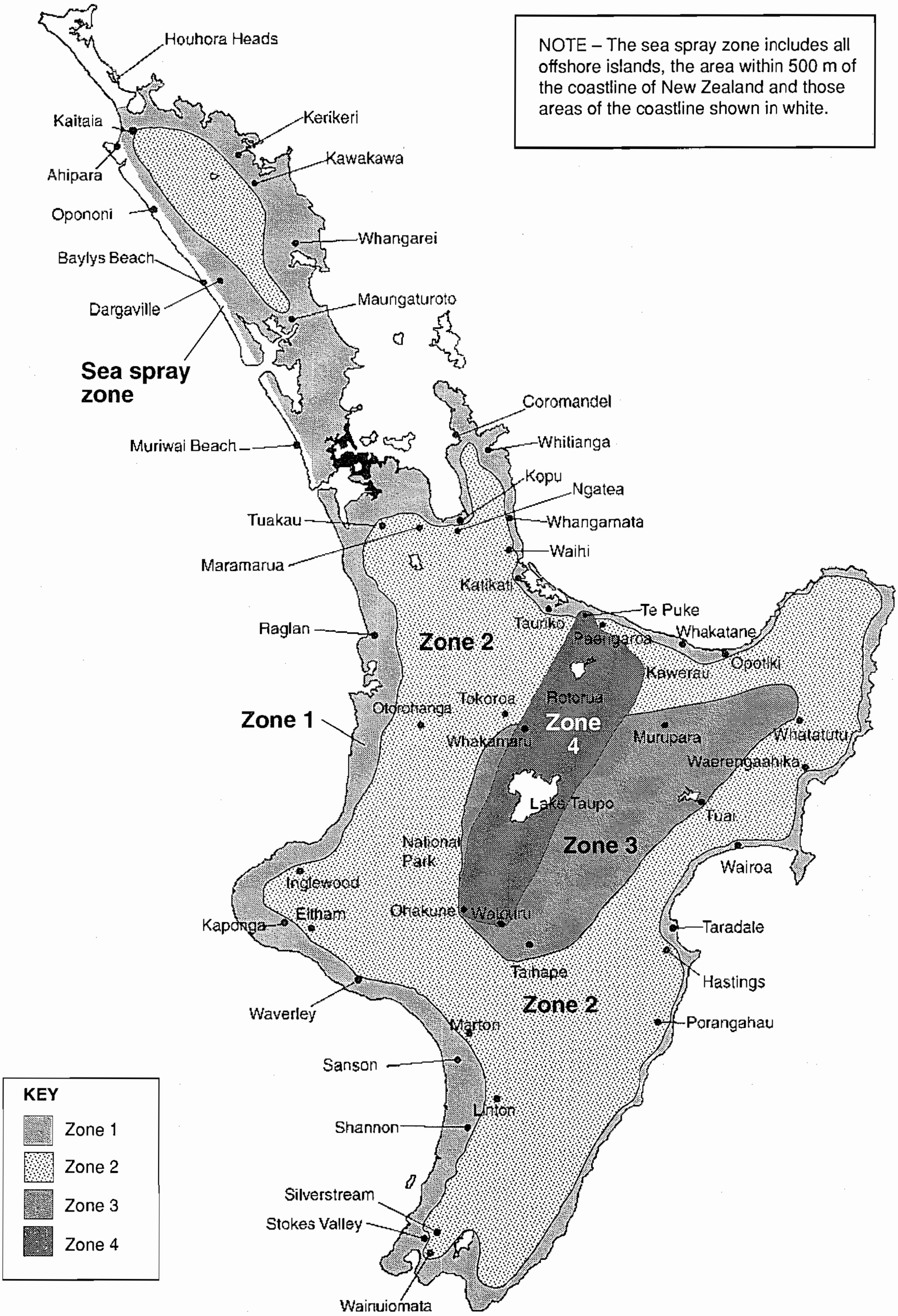
"Geothermal hot spots" are mainly found in Zone 4 but may occur elsewhere. These are areas within 50 m of a bore, mudpool, steam vent, or other fume source.

4.2.1.3 *Corrosive atmosphere*

Localized areas subject to corrosive industrial atmospheres are outside the scope of this Standard.

4.2.2 *Durability specification*

Table 4.1 shall be used to select the masonry durability specification requirements.



NOTE – The sea spray zone includes all offshore islands, the area within 500 m of the coastline of New Zealand and those areas of the coastline shown in white.

Figure 4.1 – Sea spray and corrosion zone map

NOTE – The sea spray zone includes all offshore islands, the area within 500 m of the coastline of New Zealand and those areas of the coastline shown in white.

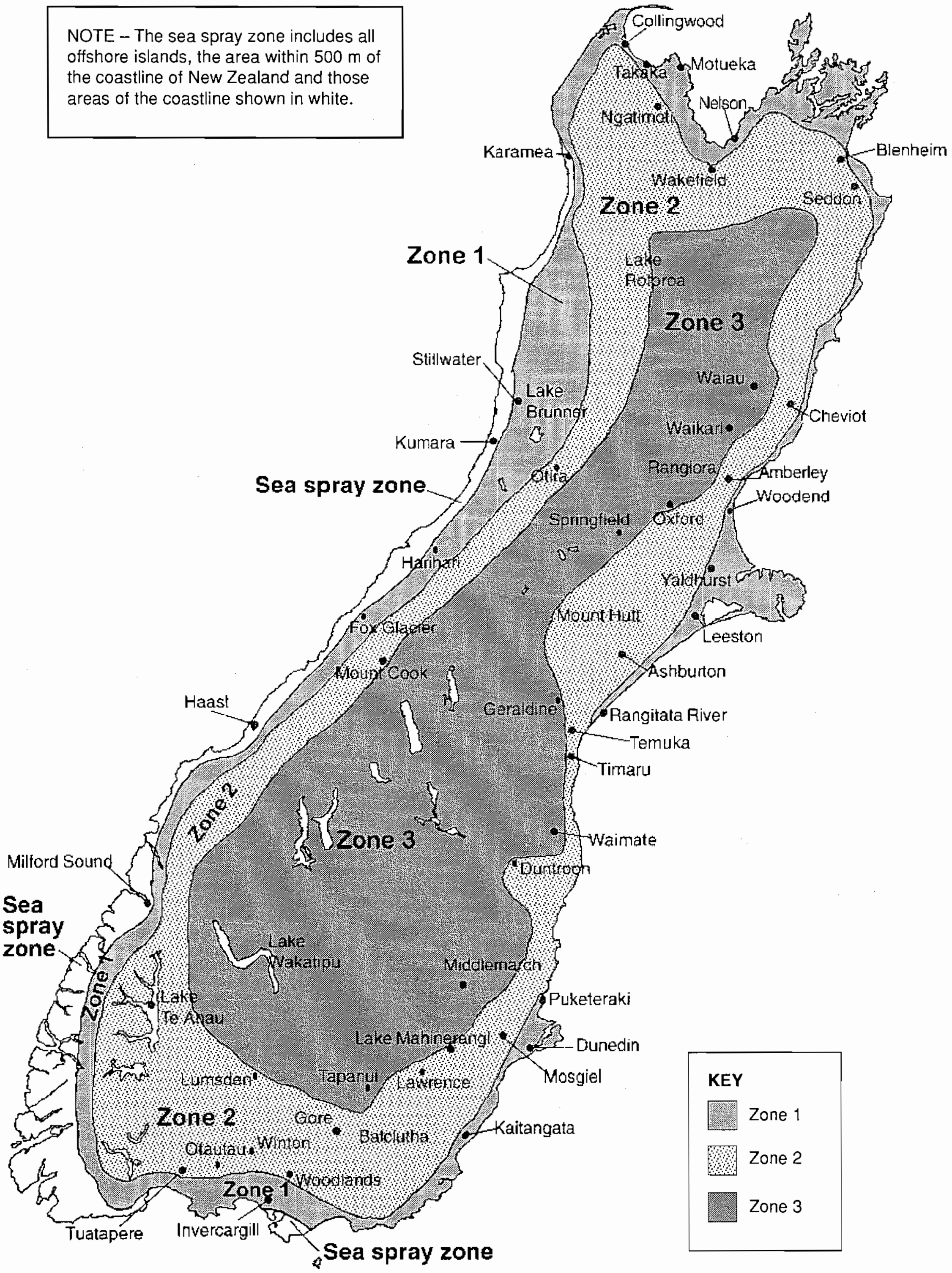


Figure 4.1 – Sea spray and corrosion zone map (continued)

C4.2

The provisions for determining exposure category zones are derived from NZS 3604.

This clause presents a simplified and conservative solution. If the corrosion exposure zone, determined for a particular site from this clause, appears to be too severe, then the designer may reclassify the site. Such reclassification however would be outside the scope of this Standard and would be an alternative solution. The alternative solution would need to be submitted to, and approved by the territorial authority as part of the building consent process.

To assist the territorial authority in its assessment of an alternative solution, it is suggested that the applicant would need to elaborate on all their considerations and in particular would need to address the following issues:

In sea spray zones:

(a) Direction of prevailing wind from the sea;

(b) Prevalence of breaking surf;

(c) Existence of salt spray residue on windows or cladding of adjacent buildings;

(d) Existence of a constant smell of salt in the air;

(e) Wind classification of site.

In Zone 1 areas:
Shelter provided by ridges or spurs, large belts of trees or other such features.

In Zone 4:
Location of building in relation to geothermal hot spots and prevailing wind.

In all zones:
Performance of adjacent buildings.

Table 4.1 – Masonry durability requirements

Exposure categories		Durability requirements	
Exposure zones	**NZS 3101 exposure classifications** (Note 1)	**Classification of built-in components** (Note 2)	**Minimum cover to reinforcement** (Note 3)
Sea spray	B2	R4	60 (30)
1 & 4	B1	R3	50 (20)
2 & 3	A2	R3	45 (15)
Closed interior (Note 4)	A1	R1	35 (5)
Geothermal hotspot	U	R5	Special study

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NOTE –

(1) The NZS 3101 zones shall be as defined in that Standard.

(2) The classifications are defined in AS/NZS 2699:Part 2 *Connectors and accessories.* A protection specification is given for the component. The manufacturer must meet this and the component must be labelled to identify the level of corrosion protection.

(3) Cover is measured from the outside of the cell face of the unit. The figures in brackets are the approximate total cover to the inside face of the wall assuming a face-shell thickness of 30 mm. Reinforcement shall be restrained so that the minimum covers are maintained during construction. Retaining walls shall be classed as B2 as specified in NZS 3101.

(4) When weatherproofed to the requirements of NZS 4210, Exposure Categories 1, 2, 3 & 4 (NZS 3604) or B1 & A2 (NZS 3101) can be reduced to "Closed Interior" or "A1". When waterproofed to the requirements of NZS 4210 all exposure categories can be reduced to "Closed Interior" or "A1".

4.3 *Governing reinforcing cover requirements*

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The requirements of section 5 for fire shall take precedence over masonry covers determined from section 4 where the required cover for durability is less than that required for fire resistance.

5 DESIGN FOR FIRE RESISTANCE

5.1 Notation

A_g	Gross area of section, mm^2
b	Thickness of a wall, mm
b_c	Minimum width of column section, mm
b_w	Effective web width, mm
c	Cover to longitudinal reinforcement, mm
E_r	Ratio of modulus of elasticity at design temperature T to modulus of elasticity at 23 °C
f'_m	Specified compressive strength of masonry, MPa
f_r	Ratio of compressive strength of concrete (or yield stress of reinforcing or prestressing steel) at design temperature T to compressive strength of concrete (or yield stress of reinforcing or prestressing steel) at 23 °C
h_{we}	Effective height of a wall, mm
h_{wu}	Unsupported height of a wall, mm
L_L	Distance between centres of lateral restraints, mm
N^*	Applied design axial load at the ultimate limit state (exclusive of selfweight), N
T	Temperature, °C
t_w	Wall thickness, mm
ϕ	Strength reduction factor, see 3.4.7

5.2 Scope

The provisions of this section set out the requirements for the design of reinforced masonry structures and components to resist the effects of fire, and gives methods for determining the fire resistance ratings required by the NZBC.

C5.2

This section is largely extracted from AS 3600, with changes to reflect the properties of New Zealand masonry where these have been adequately tested.

Terminology has been changed to conform with that of the NZBC.

In building regulations, the specification of various fire resistance levels in relation to standard fire test conditions ensures that relatively higher or lower levels of fire resistance are achieved by various types of construction. This section gives rules whereby masonry components can be proportioned and detailed to satisfy regulatory requirements for particular fire resistance levels.

The term "fire resistance rating" refers to the level of fire resistance that will be required for the structural component by the building regulations. In the NZBC, the fire resistance rating is expressed in minutes, in the order Structural Stability/Integrity/Insulation. Thus a FRR of 90/90/90 means that the structural components are required to have a resistance period for structural stability of 90 minutes, integrity of 90 minutes and thermal insulation of 90 minutes, i.e. the minimum times that would need to be achieved if the components were tested for these criteria in accordance with AS 1530:Part 4.

AS 1530:Part 4 specifies conditions for the assessment of the fire resistance of a building component or component. A prototype specimen is tested in a furnace which is operated so that the furnace temperature-time relationship is as shown in figure 5.1. The standard fire resistance test provides an internationally accepted basis for the assessment of the relative degree of fire resistance of different materials and building components.

It must be emphasized that the temperature versus time relationship corresponding to an actual fire is dependent on many factors, including quantity and type of fuel, fuel geometry, ventilation and other compartment characteristics and is likely to be significantly different from the standard time/temperature curve in a standard fire test. Furthermore, in an actual fire, unaffected portions of a building apply constraints on components which are almost impossible to simulate in a prototype test situation.

It is important to realize that AS 1530:Part 4 specifies not only a particular temperature-time relationship but also specifies the direction from which prototype test components are to be heated in the furnace, namely; floor and roof assemblies (slabs and beams) from below, walls from either sides but not both sides simultaneously and columns from all vertical sides. These requirements have important consequences on the manner in which subsequent clauses in this section are framed and the application of these clauses to the design of components in actual buildings.

Provided that the implications of these limitations are taken into account when interpreting regulatory requirements, the use of the standard fire test will remain an important component of building regulations.

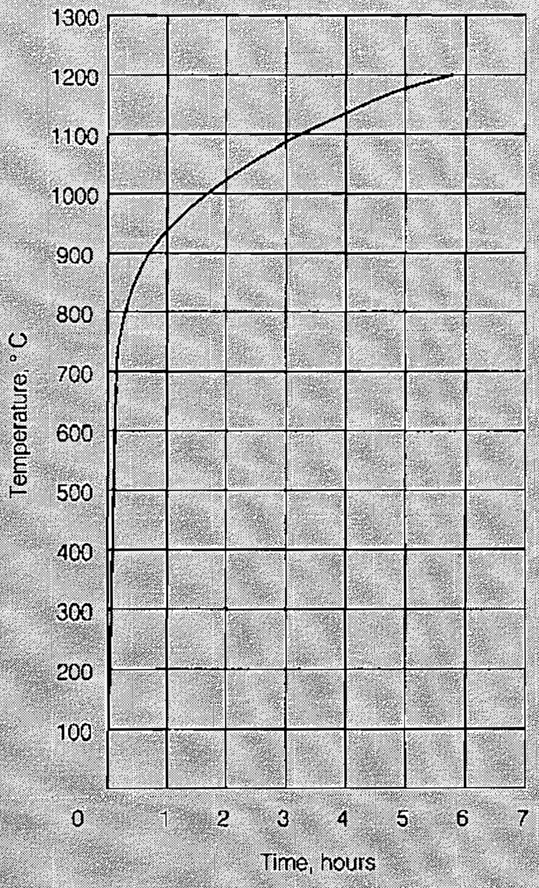

Figure 5.1 – Standard furnace temperature-time curve

5.3 Design requirements

5.3.1 *General*

A component shall be designed to have the required fire resistance rating for each of stability, integrity, and insulation. The requirements of section 4 for durability shall take precedence over masonry covers determined from this clause where the required cover for fire resistance is less than that required for durability.

C5.3.1

In designing boundary elements of a fire compartment for fire resistance, it may be assumed that such elements are exposed to fire from only one direction at a time for the purposes of interpreting this clause.

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5.3.2 *Joints*

Joints between components or between adjoining parts shall be constructed so that the fire resistance rating of the whole assembly is not less than that required for the component.

C5.3.2

Fire resistance properties of joints between masonry wall panels are described in Reference 5.1.

5.3.3 *Methods for determining fire resistance ratings*

The fire resistance ratings for a component shall be determined by either:

(a) Proportioning the component in accordance with 5.4 to 5.6, as appropriate; or

(b) The methods given in 5.7 and 5.8.

C5.3.3

Three methods of determining fire resistance ratings are given in this clause.

Clauses 5.4 and 5.5 define geometries which result in components which are deemed to satisfy the stated fire resistance ratings. The recommended cross-section dimensions and cover have been based on British and European recommendations (References 5.2, 5.3, 5.4, 5.5 and 5.6). A comparison of the various overseas recommendations with the available fire test data has been given in References 5.7, 5.8 and 5.9.

The recommended minimum cross-section dimensions and covers set out in 5.4 to 5.6 assume that the component is loaded to give an internal action of about 50 % of the ultimate capacity of the component. This simulates the fire-test situation.

5.3.4 *Loads to be considered simultaneously with fire*

All loads that are required to be considered simultaneously with fire shall be taken at the ultimate limit state unless specifically noted otherwise.

5.4 Fire resistance ratings for walls

5.4.1 *General*

The fire resistance ratings for a wall shall be determined in accordance with either:

(a) Clause 5.4.2 to 5.4.4 if the wall has a fire-separating function; or

(b) Clause 5.6 in all other instances.

5.4.2 *Insulation for walls*

A wall has a fire resistance rating for insulation given by table 5.1 if the effective thickness of the wall is not less than the corresponding value given in the table. The effective thickness of the wall to be used in table 5.1 shall be taken as follows:

(a) For solid walls, the actual thickness;

(b) For partially filled walls the net cross-sectional area divided by the length of the cross-section.

Table 5.1 – Minimum wall thickness for fire resistance ratings for insulation

Fire resistance rating (minutes)	**Effectiveness thickness for different aggregate type** (mm)		
	Type A aggregate	**Type B aggregate**	**Type C aggregate**
30	50	45	40
60	75	70	55
90	95	90	70
120	110	105	80
180	140	135	105
240	165	160	120

NOTE –
Aggregate types:
A – quartz, greywacke, basalt & and all others not listed
B – dacite, phonolite, andesite, rhyolite, limestone
C – pumice and selected lightweight aggregates.

5.4.3 *Integrity for walls*

A wall has the stated fire resistance rating for integrity if it meets the requirements for both insulation and stability for that rating.

5.4.4 *Stability of walls*

A laterally supported wall has the required fire resistance rating for stability if (a) to (d) of the following are satisfied:

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(a) Complies with the dimensional limitations, axial forces and strength requirements for walls in NZS 3101;

(b) The effective thickness of the wall is not less than the thickness required by 5.4.2 for that rating;

(c) If $N^* \leq 0.03 f'_m A_g$, and h_{we}/t_w is not greater than 50;

(d) If $N^* > 0.03 f'_m A_g$,

(i) h_{we}/t_w is not greater than 20; and

(ii) The cover from the fire-exposed face to the vertical reinforcement or tendons is not less than the corresponding cover given in table 5.2 for that rating.

For the purpose of (c) and (d) above, the following apply:

(iii) N^* is the design axial load for the ultimate limit state (exclusive of self weight) at the mid-height of the wall;

(iv) If the wall is laterally supported top and bottom only, h_{we} shall be taken as:

1.0 h_{wu} if neither support is rotationally restrained;

0.85 h_{wu} if one support is rotationally restrained; or

0.70 h_{wu} if both supports are rotationally restrained,

where the rotational restraint at the support, if any, is provided by a component outside the fire compartment (including a continuation of the wall itself).

(v) If the wall is laterally supported on all 4 sides, h_{we} shall be determined:

In accordance with (iv) if $h_{wu} \geq L_L$; or

By substituting L_L for h_{wu} in (iv) if $h_{wu} > L_L$, the rotational restraint provided being determined for the supports in the direction of L_L.

Table 5.2 – Minimum cover to vertical reinforcement and tendons for stability of walls

Fire resistance rating (minutes)	**Cover, *c*** (mm)	
	To reinforcement	**To tendons**
30	20	30
60	20	30
90	35	30
120	40	30
180	45	35
240	50	50

C5.4.4

The minimum cover to vertical reinforcement for stability of walls is given in table 5.2. The smaller values for tendons, for the higher fire resistance ratings, result from the beneficial effect of prestressing on the load-curvature behaviour of walls heated on one face.

Most compartment boundary walls, such as those associated with service cores in buildings, should be regarded as having a fire separating function and therefore being subjected to fire on one side only. This also applies to walls incorporating openings, where the wall should be proportioned with regard to stability on the assumption that the opening is fitted with an appropriate fire resistant door or window.

In multistorey buildings, fire is assumed to be confined to a single compartment bounded by fire resistant walls and floors. Under these circumstances, significant rotational restraint will be provided to the wall by the adjacent components and the cooler parts of the wall located above and below the section of wall subject to fire.

5.4.5 *Increasing fire resistance ratings for walls by insulating materials*

5.4.5.1

For walls the fire resistance ratings may be increased, in accordance with 5.9, by the application of insulating materials to the face exposed to fire.

5.4.5.2

Other methods (e.g. addition of insulation materials in hollow cores) may be used, but any increase afforded shall be determined in accordance with 5.7.

5.5 Fire resistance ratings for beams

5.5.1 *Insulation and integrity for beams*

Fire resistance ratings for insulation and integrity are not generally relevant to beams, but where required, are met by satisfying the corresponding fire resistance ratings for walls.

C5.5.1

There are instances where beams form part of a component performing a fire-separating function, e.g. spandrel or lintel beams. In such cases insulation and integrity are relevant and it is recommended that the web of the beam be considered as a wall for this purpose.

5.5.2 *Stability for beams incorporated in roof or floor systems*

A beam, whose upper surface is integral with, or protected by a reinforced concrete slab complying with NZS 3101 has one of the fire resistance ratings for stability shown in table 5.3 and table 5.4 if it is proportioned so that:

(a) The beam width, measured at the centroid of the lowest level of longitudinal bottom reinforcement; and

(b) The cover to the longitudinal bottom reinforcement are not less than the value for that rating obtained from:

  (i) Table 5.3 for simply supported beams; or

  (ii) Table 5.4 for continuous beams.

Table 5.3 – Stability requirements – Simply supported beams

Fire rating (minutes)	**Effective web thickness, b_w**		
	240	**190**	**140**
	Cover (mm)		
180	70	–	–
120	45	55	–
90	33	35	45
60	20	22	25

Table 5.4 – Stability requirements – Continuous beams

Fire rating (minutes)	**Effective web thickness, b_w**		
	240	**190**	**140**
	Cover (mm)		
180	70	–	–
120	25	35	–
90	20	20	25
60	20	20	20

For the purpose of this clause, a beam shall be considered continuous if, under imposed load, it is flexurally continuous at least at one end.

C5.5.2

This clause is concerned with the situation of floor beams heated from below.

Both the beam width and the cover to the longitudinal bottom reinforcement or to tendons are important in restricting the rise in temperature in longitudinal reinforcing bars or tendons. As the temperature of a longitudinal bottom bar increases, its strength decreases and this leads to a reduction in component strength.

Figure 5.2 in this Standard shows the reduction in strength of reinforcing and prestressing steel with increasing temperatures. It should be noted that the decrease in strength of prestressing steel, for a given rise in temperature, is greater than for Grade 300 or 500 reinforcing steel and this is the reason for the increased cover requirements for components reinforced with tendons, except in the case of prestressed walls.

Smaller covers for given levels of fire resistance are required for continuous beams than for simply supported beams. Alternatively a continuous component will attain a higher fire resistance rating compared with a simply supported component with the same amount of cover.

With fire exposure from beneath, the underside of the beam expands more than the top. This differential expansion causes the ends of the beam to tend to lift from the outer supports thus increasing the reaction due to gravity loads at the interior support. This results in a change of action in components, i.e. the negative moment at the interior support increases while the positive moment in the midspan region decreases.

During the course of a fire, changes in moments continue to occur, along with reduction of both positive and negative moment strengths. The positive moment strength is reduced because of the reduction of the strength of the bottom reinforcement as its temperature increases, whereas the negative moment strength is reduced due to concrete at the bottom of the beam heating up with associated reduction in strength. However, the positive moment strength is reduced more rapidly than the negative moment strength, hence higher temperatures can be tolerated in the bottom steel for a continuous beam, compared with a simply supported beam, before collapse occurs.

With the expansion of the beam and reduction in negative bending strength at the supports, it is possible that a plastic hinge may be formed at interior supports. Component collapse occurs when the maximum positive bending moment exceeds the positive bending strength.

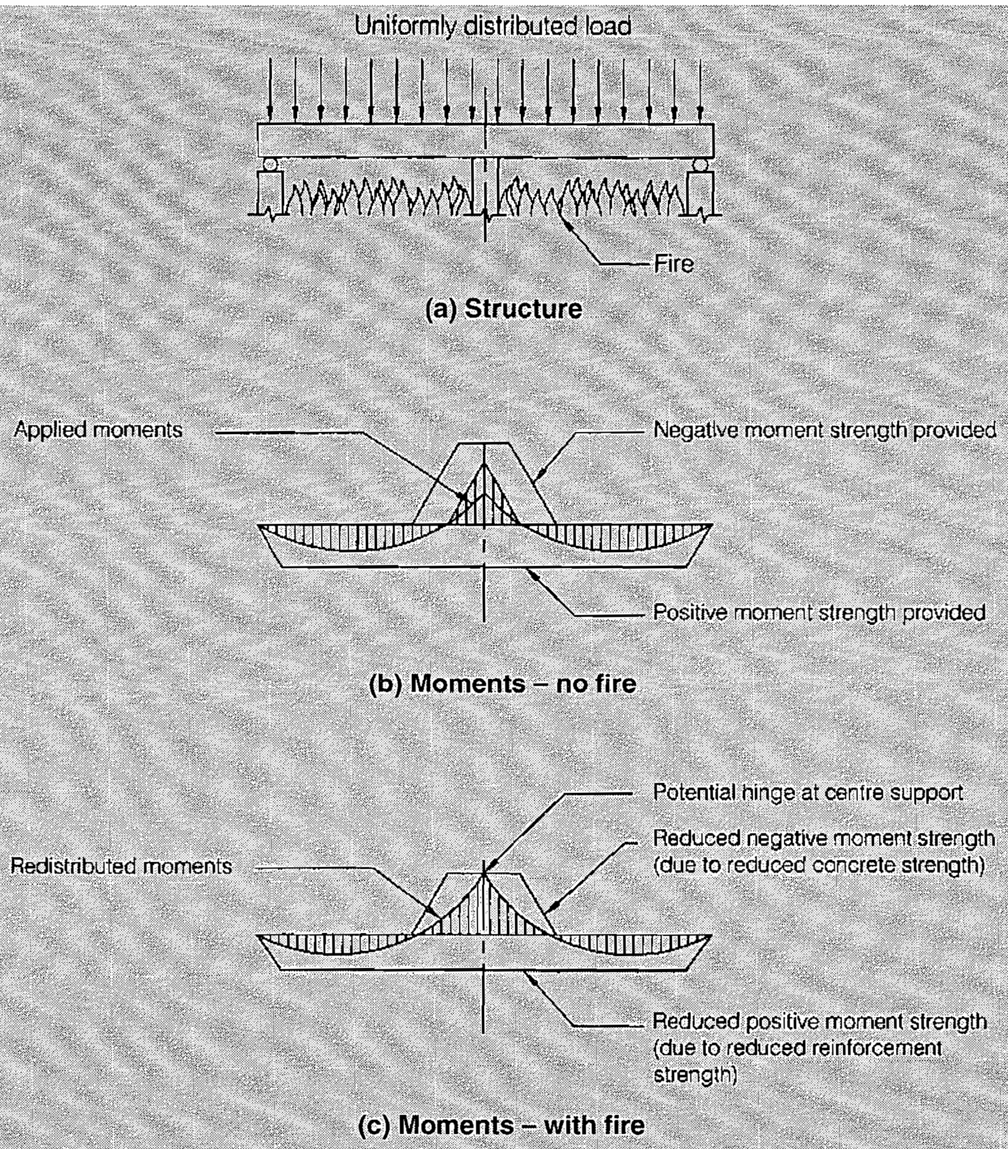
Uniformly distributed load
Fire
(a) Structure
Applied moments
Negative moment strength provided
Positive moment strength provided
(b) Moments – no fire
Potential hinge at centre support
Reduced negative moment strength (due to reduced concrete strength)
Redistributed moments
Reduced positive moment strength (due to reduced reinforcement strength)
(c) Moments – with fire

Figure 5.2 – Changes to moments in continuous beams during a fire

5.5.3 *Stability for beams exposed to fire on all sides*

A beam of rectangular cross-section which can be exposed to fire on all 4 sides has a particular fire resistance rating for stability if it is proportioned so that:

(a) The total depth of the beam is not less than the least value of b_w for that rating obtained from table 5.3 or table 5.4, as appropriate;

(b) The cross-sectional area of the beam is not less than twice the area of a square with a side equal to b_w determined as for item (a); and

(c) The cover is not less than the value for that rating determined using the minimum dimensions of the beam for b_w in the relevant figure, and applies to all longitudinal reinforcement or tendons.

C5.5.3

The particular requirements of this clause are derived from the CEB/FIP recommendations [5.5] *for beams exposed to fire on all sides. The requirements recognize that higher temperatures will be experienced by the top surface of a beam subject to fire on all sides, compared with one heated only from below.*

5.5.4 *Increasing fire resistance ratings of beams by insulating materials*
For beams, the fire resistance ratings may be increased, in accordance with 5.9, by the application of insulating material to the surfaces exposed to fire.

C5.5.4
The important point to note is that the increase is obtained by adding the insulation to the faces exposed to the fire, i.e. for T- and L- beams, to the sides of the web and the soffit.

5.6 Fire resistance ratings for columns

5.6.1 *General*
Fire resistance ratings for a column shall be determined in accordance with either:

(a) Clauses 5.6.2 and 5.6.3 if the column:

  (i) Can be exposed to fire on all sides; or

  (ii) Is built into or forms part of a wall not capable of having a fire separating function; or

  (iii) Is built into or forms part of a wall having a fire separating function but which has a fire resistance rating for stability less than that required for the column; or

  (iv) Is built into and protrudes by more than the cover to the longitudinal steel beyond the fire-exposed face of a wall having a fire-separating function; or

(b) Clause 5.4 in all other instances.

C5.6.1
The minimum dimensions and covers given in figure 5.3 are based on the assumption that the columns are exposed to fire on all sides. If narrow columns can be exposed to fire on all sides, they must be treated as columns not walls.

A column fully incorporated into a wall that is subject to heat flow from one direction only, is required to satisfy the structural stability requirements for loadbearing walls because it serves as a fire separating function.

5.6.2 *Insulation and integrity for columns*
Fire resistance ratings for insulation and integrity do not apply to columns described in 5.6.1(a). Where a column serves a fire-separating function and 5.6.1(a) is not applicable, the fire resistance ratings for insulation and integrity shall be determined in accordance with 5.4.2 and 5.7.3.

5.6.3 *Stability for columns*

5.6.3.1
A column has a fire resistance rating for stability shown in figure 5.3 if it is proportioned so that the minimum cross-sectional dimensions and cover to the longitudinal reinforcement are not less than the values obtained from that figure.

5.6.3.2
For a particular fire resistance rating, cover to tendons shall be 10 mm greater than the relevant values for longitudinal reinforcement obtained from figure 5.3.

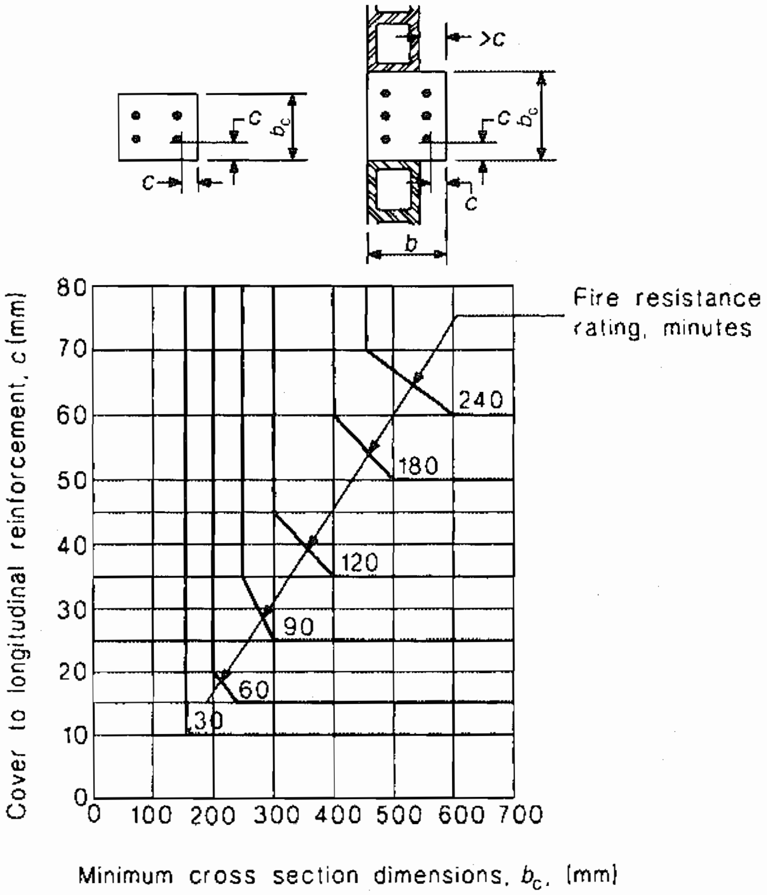

Figure 5.3 – Stability requirements – Columns

5.6.4 *Increasing fire resistance ratings for columns by insulating materials*

For columns, the fire resistance ratings may be increased, in accordance with 5.9, by the application of insulating material to the faces exposed to fire.

5.7 Fire resistance ratings from fire tests

5.7.1 *General*

Fire tests on components shall be carried out in accordance with AS 1530:Part 4 and BS 476: Parts 20-22 and the results applied in accordance with this clause, as appropriate.

5.7.2 *Loadbearing components tested under load*

C5.7.2

This clause sets out rules for the interpretation and application of the results from a fire test on a loadbearing component subjected to a test load, which may be different from the design load.

5.7.2.1 *Application of test results*

For prototype loadbearing components tested under load in accordance with AS 1530:Part 4 and BS 476:Parts 20-22, the fire test results shall be applied in accordance with 5.7.2.2.2 as appropriate.

5.7.2.2 *Components identical to the prototype*

5.7.2.2.1

The results of fire tests on a prototype may be applied directly to an identical component or system incorporated in a building structure.

5.7.2.2.2

For the purpose of 5.7.2.2.1, an incorporated component or system shall be considered identical to the prototype if:

(a) It is of the same shape, size and form of construction as the prototype;

(b) It is composed of materials having relevant properties within the variability range of those used in the prototype;

(c) It has the same type and similar degree of flexural restraint and restraint against thermal movements as the prototype;

(d) It has thickness and covers, in relation to the expected direction of fire exposure, not less than the corresponding thicknesses and covers of the prototype;

(e) It has an effective span, or effective length, which does not exceed that of the prototype by more than 3 %; and

(f) It has an applied loading resulting in peak flexural demands at the ultimate limit state no greater than those of the prototype.

5.7.2.2.3

For the purpose of this clause, an incorporated component or system shall be considered similar to the prototype component or system provided that:

(a) The incorporated component or system:

  (i) Is similar in geometry to the prototype;

  (ii) Is of the same form of construction and is composed of materials similar to those used in the prototype; and

  (iii) Has the same type of restraints against flexural or thermal movements, or both, as provided or induced in the prototype; and

(b) The calculated stresses in the incorporated component or system, due to the short-term serviceability limit state loads specified in AS/NZS 1170 do not exceed by more than 20 % the calculated stresses in the corresponding sections of the prototype, due to the loads on it at the commencement of heating.

5.7.3 *Beams, and columns tested as non-loaded components*

Temperatures measured within the cross-section of beams, and columns, tested as non-loaded components in accordance with AS 1530:Part 4 and BS 476:Parts 20-22, shall be used, in conjunction with a method of calculation given in 5.8, to determine the stability of the constructed component.

C5.7.3

AS 1530.4 allows the determination of the stability of a loadbearing component by measuring the variation with time of temperatures at critical locations in the cross-section and then using a recognized method of calculation (see C5.8 to determine the strength at any time using that record of temperatures).

5.8 Fire resistance rating by calculation

The fire resistance rating of a component may be predicted by a recognized method of calculation such as that given in BRANZ Technical Recommendation No. 8, using the load combinations given in AS/NZS 1170, a ϕ factor in accordance with 3.4 and appropriate values for the properties of New Zealand materials obtained from figure 5.4 and figure 5.5.

C5.8

Recognized procedures for calculating the behaviour of concrete masonry components in fire are those outlined in References 5.5, 5.10, 5.11, 5.12, 5.13 and 5.14. Such a calculation involves the determination of temperatures within the concrete cross-section and then the evaluation of associated component strength.

The equations on which the figures in the Standard are based are given with the corresponding lines in figure 5.4 and figure 5.5.

For calculating the structural integrity of concrete components, the strength reduction factor ϕ and the applied load factors shall be 1.0. A factor of safety for fire design is included in the fire resistance rating.

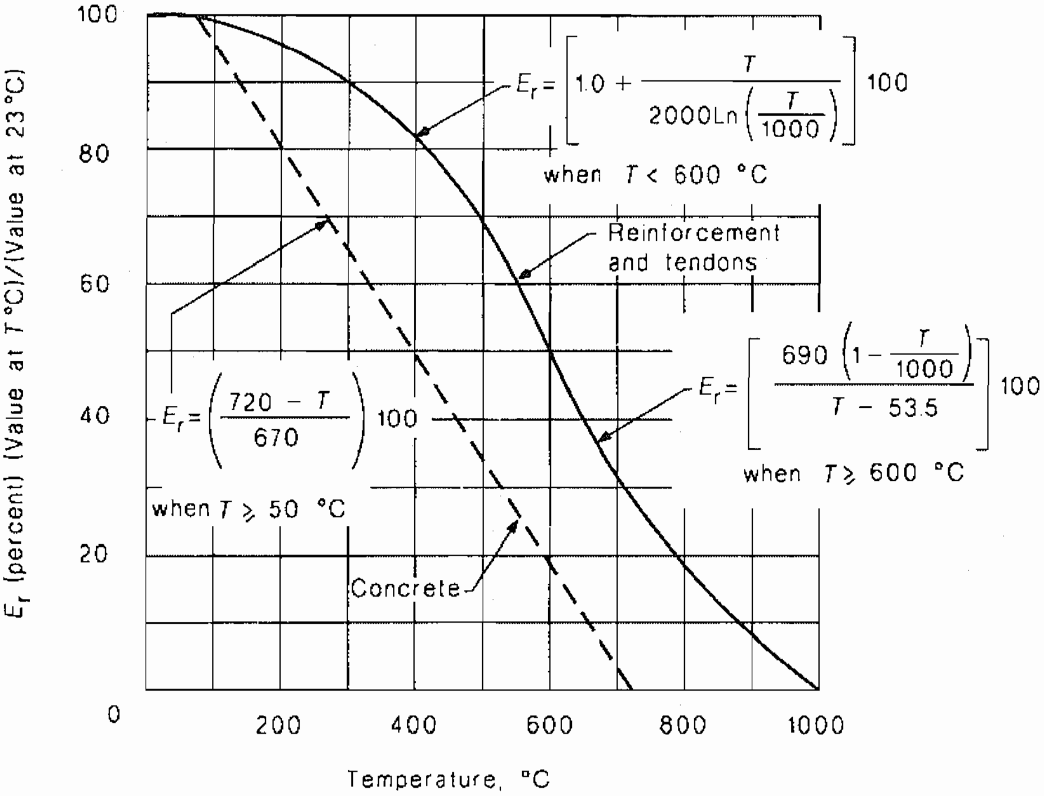

Figure 5.4 – Values of elastic modulus to be used in determining fire resistance rating by calculation

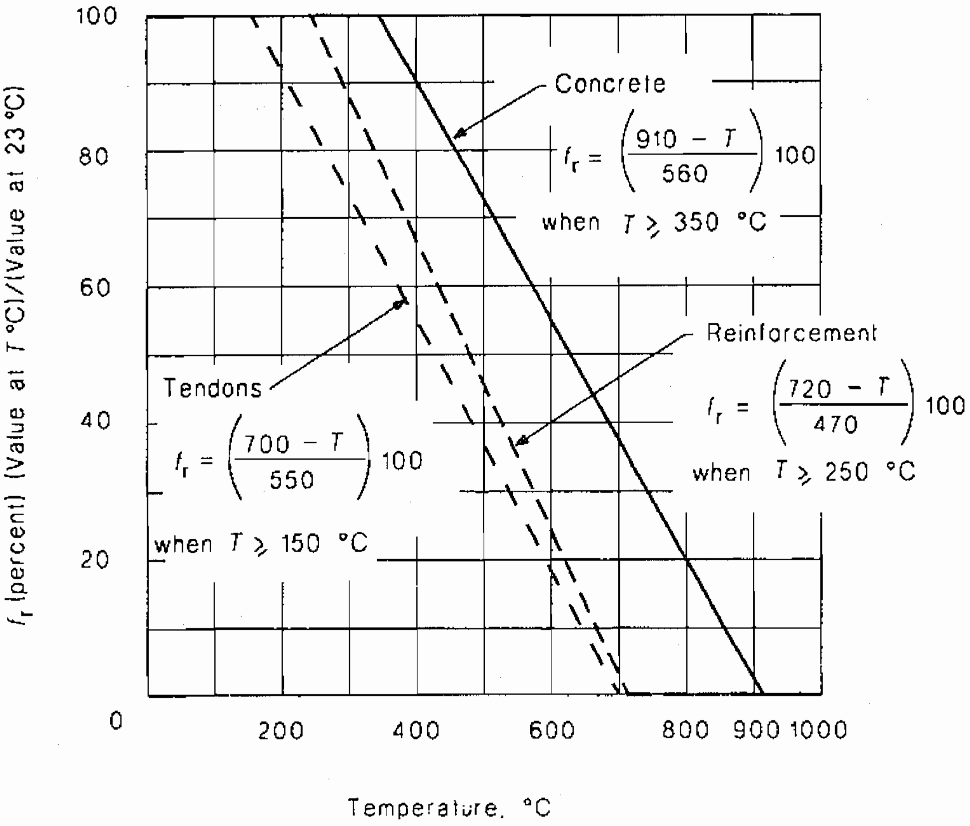

Figure 5.5 – Values of strength to be used in determining fire resistance rating by calculation

5.9 Increase of fire resistance ratings by use of insulating materials

5.9.1 *Increase of fire resistance ratings by the addition of insulating materials*

5.9.1.1 *General*

The fire resistance ratings for insulation and stability of a masonry component may be increased by the addition to the surface of an insulating material to provide increased thickness to the component, or greater insulation to the longitudinal reinforcement or tendons, or both.

C5.9.1.1

The placing of a topping on top of a slab will increase the fire resistance rating with respect to insulation only. The application of an insulation material to the side(s) of the component closest to the fire results in lower steel and concrete temperatures for a given time of fire exposure, and consequently increases fire resistance ratings with respect to both insulation and stability.

While the addition of insulation is straightforward for slabs and beams in floor or roof systems, careful thought needs to be applied in determining the proper positioning of the additional material on sides of column components and walls.

Except for walls forming the sides of a fire-isolated passageway or shaft, fire is assumed to be on either face of the wall, irrespective of whether it is an interior or an exterior wall.

If the deficiency in the wall relates only to insulation, the additional material can be added to either or both faces, since effective thickness is the only criterion in this case. The decision here will therefore depend only on ease of application.

If the deficiency relates to stability, cover to the reinforcement is the criterion, so the additional material will need to be added to the face for which the cover to the reinforcement is deficient, i.e. both faces for a centrally reinforced wall and one or both faces, as appropriate, for a doubly reinforced wall. In the case of a wall forming the sides of a fire-isolated passageway or shaft, the fire is always assumed to be outside the passageway or shaft, hence the additional material would need to be placed on the face opposite to the interior but only if the cover on that particular face is deficient.

5.9.1.2 *Acceptable forms of insulation*
Acceptable forms of insulation include the following:

(a) Thicknesses of 1:4 vermiculite concrete or of 1:4 perlite concrete which are appropriately bonded to the concrete masonry;

(b) Gypsum-vermiculite plaster or gypsum-perlite plaster, both mixed in the proportion of 0.16 m^3 of aggregate to 100 kg of gypsum, in the form of either thickness added and appropriately bonded to the concrete, or as a sprayed or trowelled application applied *in situ*;

(c) Any other fire protective building board or material that has been demonstrated to be suitable for the purpose in a standard fire resistance test.

5.9.1.3 *Thickness of insulating material*

5.9.1.3.1
The minimum thickness of insulating material added to attain the required fire resistance level shall be determined by testing in accordance with AS 1530:Part 4 and BS 476:Parts 20-22.

5.9.1.3.2
In the absence of such testing and only for the materials specified in 5.9.1.2, the minimum thickness of insulating material to be added may be taken as the difference between the required cover or effective thickness specified in this section and the actual cover or effective thickness, whichever governs, multiplied by:

(a) 0.75, for materials specified in 5.9.1.2 (a) and (b); or

(b) An appropriate factor for materials specified in 5.9.1.2(c), where the factor is derived from tests in which the difference calculated above lies within the range of insulation thicknesses tested; and

(c) The thickness thus calculated rounded to the nearest 5 mm above.

5.9.1.4 *Reinforcement in sprayed or trowelled insulating materials*
Where the thickness of sprayed or trowelled insulating materials exceeds 10 mm, the material shall be reinforced to prevent detachment during exposure to fire.

REFERENCES

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6 REINFORCEMENT

6.1 Notation

A_{sp}	Area of flexural reinforcement provided, mm^2
A_{sr}	Area of flexural reinforcement required, mm^2
A_t	Area of bar formed into spiral or circular hoop reinforcement, mm^2
A_{tr}	Smaller of area of transverse reinforcement within a spacing, s, crossing plane of splitting normal to masonry surface containing extreme tension fibres, or a total area of transverse reinforcement normal to the layer of bars within a spacing, s, divided by n, mm^2. If longitudinal bars are enclosed within spiral reinforcement, $A_{tr} = A_t$, mm^2
A_v	Area of shear reinforcement within a distance, s, mm^2
d	Distance from extreme compression fibre to centroid of tension reinforcement, mm
d_b	Nominal diameter of bar, wire, or in a bundle, the diameter of a bar of equivalent area, mm
d_i	Diameter of bend measured to the inside of the reinforcing bar, mm
f_y	Lower characteristic yield strength of non-prestressed reinforcement, MPa
f_{yt}	Lower characteristic yield strength of transverse reinforcement, MPa
L_d	Development length, mm
L_{db}	Basic development length of a straight bar, mm
L_{dh}	Development length of a hooked bar, equal to straight embedment between critical section and point of tangency of hook, plus bend radius, plus one bar diameter, mm
n	Number of bars in a layer
s	Maximum spacing of transverse reinforcement within L_d, or spacing of stirrups or ties or spacing of successive turns of a spiral, all measured centre-to-centre, mm
V^*	Design shear force at section at ultimate limit state, N
β_b	Ratio of area of reinforcement to be cut off, to total area of tension reinforcement at the section, including those bars which are to be cut off

6.2 Scope

Provisions of section 6 shall apply to detailing of reinforcement, including spacing, development and splices.

6.3 General principles and requirements for all structures

6.3.1 *Steel reinforcement*

6.3.1.1 *Use of deformed reinforcement*
All reinforcement other than ties, stirrups, spirals, and joint reinforcement shall be deformed.

C6.3.1.1
In general, plain round bars are preferable for ties and stirrups because the small radius bends which are required have undesirable metallurgical and mechanical effects on deformed bars. Also, in most situations ties and stirrups do not rely on high bond strengths along their straight legs for their action. However, there are some cases, such as deep beams within wall structures, where it will be necessary for stirrups and ties to develop high bond values along their straight portions. In such cases it is acceptable to use deformed bars, provided that the radii satisfy table 6.2.

6.3.1.2 *Reinforcing compliance*
Reinforcing bars and wire shall conform to AS/NZS 4671.

6.3.1.3 *Reinforcement class*
The following requirements apply when specifying reinforcement:

(a) Where ductility, moment redistribution or yielding of the reinforcement can reasonably be expected during the life of the structure, Class E reinforcement as defined in AS/NZS 4671 shall be used in the yielding region;

(b) Where different classes of reinforcement are used on one project, provision shall be made to confirm that the correct class is located in the correct location.

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6.3.1.4 *Restrictions on in-line quenched and tempered reinforcement*
Reinforcing bars manufactured by the in-line quenched and tempered process shall not be used where welding, galvanising, hot bending, or threading of bars occurs.

6.3.2 *Hooks*
The term, standard hook, see figure 6.1, as used herein shall mean either:

(a) A 180° turn plus an extension of at least 4 bar diameters but not less than 65 mm at the free end of the bar; or

(b) A 90° turn plus an extension of at least 12 bar diameters at the free end of the bar; or

(c) A stirrup hook, which is defined as a 135° turn around a longitudinal bar plus an extension at the free end of the bar embedded in the grouted core of the component of at least 8 stirrup bar diameters for plain bars and 6 stirrup bar diameters for deformed bars.

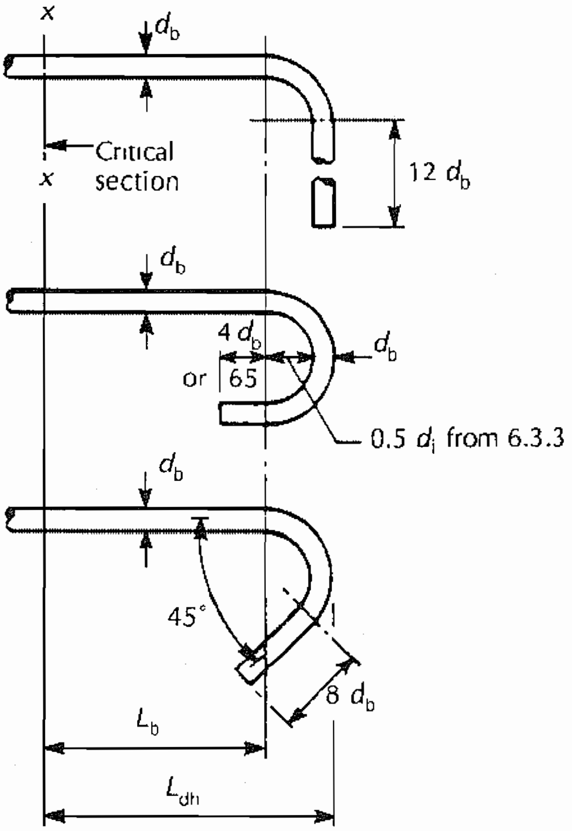

Figure 6.1 – Standard hooks

6.3.3 *Minimum bend diameter other than stirrup and tie bends*

The diameter of bend, measured to the inside of the bar, shall not be less than the appropriate value given in table 6.1.

Table 6.1 – Minimum diameters of bend for Class E steel bars to AS/NZS 4671 other than stirrup and tie bends

Steel yield strength, f_y (MPa)	**Bar diameter, d_b** (mm)	**Minimum diameter of bend, d_i**
300 or 500	6 – 20	5 d_b
	25 – 32	6 d_b

6.3.4 *Stirrup and tie bends*

Inside diameter of bends of stirrups and ties shall be equal to the diameter of the enclosed bar except that it shall be not less than the values given in table 6.2 where d_b is the stirrup or tie bar diameter.

Table 6.2 – Minimum diameters of bends for stirrups and ties for Class E steel bars to AS/NZS 4671

Steel yield strength, f_y (MPa)	**Bar diameter, d_b** (mm)	**Minimum diameter of bend, d_i**	
		Plain bars	*Deformed bars*
300 or 500	6 – 20	2 d_b	4 d_b
	25 – 32	3 d_b	6 d_b

6.3.5 *Bundled bars*

Groups of parallel reinforcing bars bundled in contact, assumed to act as a unit, and not more than two in any one bundle, may be used only when the bundle is within the perimeter of stirrups or ties. Bars larger than 16 mm shall not be bundled in concrete masonry construction.

6.3.6 *Spacing of reinforcement*

6.3.6.1 *Spacing between bars in a layer*

The clear distance between parallel reinforcing bars in a layer shall be not less than 25 mm nor the nominal diameter of the bars.

C6.3.6.1

The spacing limits of this clause have been developed from successful practice over many years, remaining essentially unchanged through many Standards. The minimum limits were established to permit block fill grout to flow readily into spaces between bars and forms without honeycombing, and to ensure against concentration of bars on a line that might result in shear or shrinkage cracking.

6.3.6.2 *Bar arrangement and spacing between layers*

Where parallel reinforcement is placed in two or more layers in beams, the bars in the upper layers shall be placed directly above those in the bottom layer with the clear distance between layers not less than 25 mm nor the nominal diameter of the bars.

6.3.6.3 *Spacing determined by aggregate size*

The nominal maximum size of the aggregate shall be not larger than 0.75 of the minimum clear spacing between the individual reinforcing bars or bundles.

6.3.6.4 *Spacing of bundled bars*

To determine the spacing limits of bundled bars, the bundle shall be treated as a single bar of a diameter derived from the equivalent total area.

6.3.7 *Development lengths of reinforcement*

6.3.7.1 *General*

Calculated tension or compression in reinforcement at each section of a reinforced concrete masonry component shall be developed on each side of that section by embedment length or end anchorage or a combination thereof. Hooks may be used in developing bars in tension.

6.3.7.2 *Development length of deformed bars and deformed wire in tension or compression*

The development length, L_d, of deformed bars shall be computed as the product of the basic development length, L_{db}, from 6.3.7.3 and 6.3.7.4 and when applicable the modification factor or factors in 6.3.7.5.

C6.3.7.2

The figures given in this clause have been extrapolated from values based on tension tests of lapped bars in prisms for Grades 275 and 380 bars, as provided in NZS 4230:1990. The figures are considered to provide a reasonable estimate of lap lengths pending the result of future testing on Grade 300 and 500 reinforcement respectively.

6.3.7.3 *Basic development length of grade 300 reinforcement*

The basic development length, L_{db}, for reinforcement having $f_y = 300$ MPa shall be 40 d_b.

6.3.7.4 *Basic development length of grade 500 reinforcement*

The basic development length, L_{db}, for reinforcement having $f_y = 500$ MPa shall be 70 d_b.

6.3.7.5 *Modification factors for basic development length*

The basic development length shall be multiplied by the applicable factor or factors for:

(a) Top horizontal reinforcement where more than 300 mm of fresh grout is cast in the component below the bar 1.3

Unless the grout contains an expansive admixture as defined in NZS 4210 .... 1.0

(b) Reinforcement in a flexural component (not subjected to seismic loads nor required for temperature or shrinkage in restrained components) in excess of that required .... A_{sr}/A_{sp}

C6.3.7.5

The modification factors specified in this clause are multiplied together when more than one is applicable. The 1.3 factor for top reinforcement recognizes the reduction in the quality of bond when the excess water used in the mix for workability and air entrapped during the mixing and placing operations rise toward the top of the finished concrete masonry before setting is complete. Entrapped below bars, this water and air leaves the bar less bonded to the grout on the underside. For horizontal top bars in a structural component, bond resistance reflects this weakened underside restraint because the loss can be of the order of 50 % in extreme cases.

Because the development length required is proportional to the tensile stress to be developed, the full development length may be reduced proportionally when the stress is lower than the yield strength. This is achieved by the modification factor A_{sr}/A_{sp}. It should be noted, however, that this reduction must not be used at or near critical sections of components subjected to earthquake loads.

6.3.7.6 *Development length of plain bars and wire in tension or compression*

The development length for plain bars and wire shall be twice the calculated value of L_d for a deformed bar.

6.3.7.7 *Development of bundled bars*

Development length of individual bars within a two-bar bundle, in tension or compression, shall be that for the individual bar, increased by 20 %.

Individual bars in a two-bar bundle cut off within the span of flexural components shall terminate at different points with at least a 40 d_b stagger.

C6.3.7.7

An increased development length for individual bars is required when two bars are bundled together. The extra extension is needed because the grouping makes it more difficult to mobilize bond resistance from the "core" between the bars. It is recommended that wherever possible bundled bars should not be used.

6.3.8 *Anchorage of reinforcement*

6.3.8.1 *Standard hooks in tension*

Where hooks are to be specified:

(a) The development length, L_{dh}, of a deformed bar in tension terminating in a standard hook shall be taken as the greater of 20 d_b or 250 mm for f_y = 300 MPa, or the greater of 35 d_b or 420 mm for f_y = 500 MPa;

(b) Hooks shall not be considered effective in developing reinforcement in compression.

6.3.8.2 *Mechanical anchorage*

Where mechanical anchorages are being considered:

(a) Any mechanical device capable of developing not less than 1.6 f_y or the breaking strength of the reinforcing bar, whichever is smaller, may be used as anchorage;

(b) The adequacy of such mechanical devices shall be established by special study or suitable tests.

6.3.9 *Splices in reinforcement*

6.3.9.1 *Lap splices*

Where lap splices are required:

(a) The minimum length for lap splices of bars in tension shall be taken equal to the development length, L_d, in 6.3.7.3 and 6.3.7.4 for deformed bars and equal to the development length in 6.3.7.6 for plain bars;

(b) Lap splices shall not be used for bars larger than 25 mm;

(c) Lap splices of bundled bars shall be based on the lap splice length required for individual bars of the same size as the bars spliced, and such individual splices within the bundle shall not overlap each other. The length of lap shall be increased by 20 % for a two-bar bundle;

(d) Bars of contact lap splices shall be tied together.

C6.3.9.1

For ductility, lap splices should be adequate to develop more than the yield strength of the reinforcement; otherwise a component may be subject to sudden splice failure when the yield strength of the reinforcement is reached and no "toughness" is obtainable in the component. The lap splice lengths specified in the Standard satisfy this ductility requirement. Where required:

(a) Splices should, if possible, be located away from points of maximum tensile stress;

(b) Lap splices of large bars are limited by the size of the flue area and width of masonry units;

(c) The increased length of lap required for bars in bundles is based on the reduction in the exposed perimeter of the bars;

(d) Bars in lap splices must be positioned in the same flue, and securely fixed together during grouting.

6.3.9.2 *Welded splices*
Where splices are to be welded:

(a) Except as provided herein, all welding of reinforcing bars shall conform to AS/NZS 1554:Part 3. In the design and execution of welding of reinforcing bars manufactured to AS/NZS 4671 appropriate account shall be taken of the process of manufacture;

(b) Welds shall not be made closer than 3 d_b from the commencement of bends.

C6.3.9.2
It is feasible to produce welds in bars with the required mechanical and metallurgical properties. However for some steels this requires a high level of competence and technical knowledge on the part of the welding fabricator and on the part of the inspection staff, together with special quality control facilities. Welds should not be made closer than 3 d_b *from bends in order to minimize the effect of strain age embrittlement.*

(Refer to NZS 3101 for further guidance on welded reinforcement splices.)

6.3.9.3 *Conditions for welded splices*
Welded splices shall satisfy the following conditions:

(a) A full strength welded splice is one in which the bars are butt welded to develop in tension the breaking strength of the bar;

(b) A high strength welded splice is one in which the bars are lap or butt welded to develop in tension the breaking strength of the bar.

6.3.9.4 *Mechanical connections*
A mechanical connection is defined as a connection which relies on mechanical interlock with the bar deformations to develop the connection capacity. Reinforcement of mechanical connection systems shall meet both strength and slippage criteria of ISO/CD 15835 for use in both static and seismic conditions.

C6.3.9.4
An overstrength factor is applied to prevent brittle failure of the mechanical connector. This overstrength factor corresponds to the values listed in 3.7.4.2.

A slippage limit of 0.1 mm is imposed to limit the width of potential surface cracks, conforming with surface crack width specifications in NZS 3101.

Verification of serviceability performance requires three cycles to 60 % of nominal tensile yield load of the spliced reinforcing bars. The use of mechanical connection systems in fatigue loading situations shall be the subject of a special study.

For seismic conditions, slippage limits shall comply with the plastic cyclic test criteria of ISO/CD 15835.

The performance of mechanical connection systems when subjected to large plastic curvatures such as occurs in beam and column plastic hinges should be verified through special study.

6.3.10 *Development of flexural reinforcement*

6.3.10.1 *General*
Tension reinforcement may be developed by bending across the web to be anchored or made continuous with reinforcement on the opposite side of the component.

6.3.10.2 *Critical sections for reinforcement development*

Critical sections for development of reinforcement in flexural components are at points of maximum stress and at points within the span where adjacent reinforcement terminates, or is bent.

6.3.10.3 *Development of flexural reinforcement*

Except at supports of simple spans and at the free end of cantilevers, tension reinforcement shall extend beyond the point at which, according to the appropriate bending moment envelope, it is:

(a) Required at full strength for a distance equal to the development length, L_d, plus the effective depth of the component; and

(b) No longer required to resist flexure for a distance of 1.3 times the effective depth of the component.

6.3.10.4 *Restrictions on termination of flexural reinforcement in tension zones*

Flexural reinforcement shall not be terminated in a tension zone unless one of the following conditions is satisfied:

(a) Shear at the cut-off point does not exceed 67 % of the shear capacity provided, including shear strength provided by shear reinforcement; or

(b) In the case of beams and columns, stirrup area in excess of that required for shear and torsion shall be provided along each terminated bar or wire over a distance from the termination point equal to 75 % of the effective depth of the component. Excess stirrup area, A_v, shall be not less than $0.4b\,s/f_y$. Spacing, s, shall be not more than $d/8\beta_b$; or

(c) For a 25 mm bar and smaller, continuing reinforcement provides double the area required for flexure at the cut-off point and shear does not exceed 75 % of that permitted.

6.3.10.5 *Anchorage of reinforcement where stress is not proportional to moment*

Adequate end anchorage shall be provided for tension reinforcement in flexural components where reinforcement stress is not directly proportional to moment, such as stepped footings or deep flexural components.

6.3.10.6 *Anchorage into columns*

When longitudinal beam bars are anchored in column cores or beam stubs the development shall be deemed to commence at the column face of entry.

6.3.11 *Concrete protection of reinforcement for durability and fire*

Amd 1 Dec '06 The minimum concrete cover provided for reinforcing bars shall be the greater of those derived in accordance with section 4 and section 5 of this Standard.

6.3.12 *Protection of exposed reinforcing bars and fittings*

Exposed reinforcing bars, inserts and plates intended for bonding with future extensions shall be galvanized or otherwise protected from corrosion.

6.4 Principles and requirements additional to 6.3 for structures designed using a limited ductile or ductile seismic design philosophy

6.4.1 *Development of flexural reinforcement for seismic loading*

6.4.1.1 *Attainment of flexural overstrength in potential plastic hinges*

For structures designed using a limited ductile or ductile seismic design philosophy, the distribution and curtailment of the longitudinal flexural reinforcement shall be such that the flexural overstrength of a section can be attained at critical sections in potential plastic hinge regions.

6.4.1.2 *Anchorage into columns*

When longitudinal beam bars are anchored in column cores or beam stubs, the anchorage shall be deemed to commence:

(a) Where potential plastic hinges form in the beams at the column face, one-half of the relevant depth of the column or 10 d_b, whichever is less, from the face at which the beam bar enters the column;

(b) Where it can be shown that the critical section of the hinge is at a distance of at least the beam depth or 500 mm, whichever is less, from the column face, the development length may be considered to commence at the column face of entry.

6.4.1.3 *Termination of tension reinforcement in potential plastic hinge zones*

Principal tension reinforcement shall not be terminated within potential plastic hinge regions in beams or columns in structures designed using a limited ductile or ductile design philosophy.

6.4.2 *Splices in reinforcement for seismic loading*

6.4.2.1 *Lap splices in potential plastic hinge zones*

Requirements for lap splices in potential plastic hinge regions of walls are detailed in 7.4.5.4. Lap splices shall not be used in potential plastic hinge regions in beams and columns.

C6.4.2.1

It has been observed experimentally that reinforcement laps in masonry which is unconfined and is subjected to cyclic alternate tension and compression strains of more than 0.003 will break down from bond failure associated with vertical splitting caused by compression strains in masonry and reinforcement. This action has been found to be independent of lap length, though with a longer lap length a greater number of cycles are required before the laps completely break down.

6.4.2.2 *Restriction on the location of splices*

Welded splices meeting the requirements of 6.3.9.3 (a) may be used in any location. For all other splices the following restrictions apply:

(a) No portion of any splice shall be located within the beam-column joint region, or within twice the effective component depth from the critical section of a potential plastic hinge in a beam;

(b) In a column the centre of the splice must be within one eighth of the storey height either side of the centre of the column unless it can be shown that potential plastic hinges cannot develop in the column adjacent to the beam faces.

6.4.2.3 *Confinement required at splices for beams and columns*

Tensile reinforcement in beams or columns shall not be spliced by lapping in a region where reversing stresses at the ultimate limit state may exceed 0.6 f_y in tension or compression unless each spliced bar is confined by stirrup ties so that:

$$\frac{A_{tr}}{s} > \frac{d_b f_y}{48 f_{yt}} \quad \text{(Eq. 6-1)}$$

7 STRUCTURAL WALLS

7.1 Notation

A_g	Gross area of section, mm^2
A_m	Usable area of masonry section, mm^2
A_p	Area of confining plate perpendicular to direction of confinement, mm^2
a	Depth of fractional compression joint, mm
b	Thickness of wall section, mm
c	Computed distance of neutral axis from the compression edge of the wall section, mm
d_b	Nominal diameter of bar, wire, or in a bundle, the diameter of a bar of equivalent area, mm
f'_m	Specified compressive strength of masonry, MPa
f_y	Lower characteristic yield strength of non-prestressed reinforcement, MPa
f_{yh}	Lower characteristic yield strength of confining plates, cross tie or stirrup tie reinforcement, MPa
h''	Dimension of confined masonry core measured perpendicular to the direction of confining plate being considered, mm
h_b	Overall depth of beam, mm
h_w	Total height of wall from base to top, mm
K	Factor used to determine the equivalent masonry stress for confined masonry, see 10.2.2.6
L_n	Clear vertical distance between lines of effective horizontal support or clear horizontal distance between lines of effective vertical support, mm
L_w	Horizontal length of wall, mm
N_{nw}	Nominal axial load-carrying capacity of a bearing wall designed to 7.3.4.8, N
N^*	Design axial compression load on wall, N
S_p	Spacing of shear reinforcement in direction parallel to longitudinal reinforcement, mm
s_h	Centre-to-centre spacing of confining plates, mm
v_g	Maximum permitted type-dependent total shear stress, defined in 10.3.2.4, MPa
v_m	Maximum permitted grade-dependent shear stress provided by masonry, defined in 10.3.2.5, MPa
v_n	Total shear stress corresponding to V_n, MPa
Z	Seismic hazard factor, specified in AS/NZS 1170
ε_u	Ultimate compression strain of masonry
ϕ	Strength reduction factor, specified in 3.4.7
μ	Structural ductility factor

C7.1

The following symbols which appear in this clause of the commentary are additional to those used in section 7.

p_s *Volumetric ratio of confining plate to confined core*

t *Thickness of confinement plate, mm*

7.2 Scope

The design of all loadbearing and shear wall structures, irrespective of the seismic design philosophy, shall comply with 7.3. Clause 7.4 provides requirements additional to 7.3 for structures that are designed assuming limited ductile or ductile behaviour in an earthquake.

7.3 General principles and requirements for all structures

7.3.1 *General design principles*

7.3.1.1 *Design loads*

Walls shall be designed for all vertical, lateral in-plane and face loadings to which they may be subjected including provision for eccentric loads.

7.3.1.2 *Design for flexure with or without axial loads*

Walls subjected to combined flexure and axial loads shall be designed in accordance with the principles of 10.2 including the effects of slenderness, except as modified by the provisions of 7.3.1.

7.3.1.3 *Design for shear and torsion*

Walls subjected to shear and/or torsion shall be designed in accordance with the principles of 10.3 except as modified by the provisions of 7.3.1.

7.3.1.4 *Beams-column joints*

Structural walls that form part of a beam-column joint shall be designed in accordance with section 11.

7.3.1.5 *Short walls subject to axial loads to be designed as columns*

For short walls:

(a) Walls that are less than 790 mm long and support a design axial load N^* greater than $0.1 \phi f'_m A_m$ shall be designed as columns to the provisions of 9.3 with a minimum thickness as defined in 9.3.3;

(b) Walls that are less than 790 mm long and support a design axial load N^* not greater than $0.1 \phi f'_m A_m$ may be designed as walls or columns to the provisions of 7.3 or 9.3 respectively.

C7.3.1.5

Masonry walls shorter than 790 mm have less dependable behaviour than larger walls, as a result of increased significance of the masonry unit size compared with wall length, and reduced number of reinforcing bars. High axial load levels are likely to emphasize this less dependable behaviour, and imply greater dependence on the element for vertical load support. Consequently such walls are required to be designed as columns to the provisions of section 9, which will require a minimum column width of 240 mm and increased levels of reinforcement.

7.3.1.6 *Effective widths for flanged shear walls*

In flanged shear walls, the flange and web shall be built integrally unless shown by a special study to be otherwise effectively bonded together, and the following shall apply:

(a) The effective width of a flange resisting stresses due to flexure shall not exceed one third of the height of the shear wall, and the effective overhanging flange width on each side of the web shall not exceed:

  (i) Eight times the flange thickness; nor

  (ii) Half the clear distance to the next web.

(b) For shear walls with a flange on one side only, the effective adjacent length of wall considered in flexural resistance shall not exceed:

  (i) One ninth of the height of the shear wall; nor

  (ii) Six times the flange thickness; nor

  (iii) Half the clear distance to the next web.

7.3.1.7 *Moment of inertia for flanged shear walls*

In computing the effective moment of inertia of cracked sections, the effective width of the overhanging parts of flanged components shall be one half of that given in 7.3.1.6.

7.3.1.8 *Effective length of walls subject to point face loads*

The length of wall to be considered as effective for face-load moments resulting from a concentrated load or reaction shall not exceed the centre-to-centre distance between loads, nor the width of bearing plus four times the wall thickness, unless the effective length is established from a special study.

7.3.1.9 *Effective widths for pilasters*

Pilasters may be designed as T-beams provided the maximum axial stress does not exceed $0.1\phi f'_m A_m$. The limitations of 8.3.5 and 8.4 shall apply.

7.3.2 *Minimum grout space*

Minimum grout space shall comply as follows:

(a) Walls shall be constructed such that there is a minimum clear grout space of 60 mm, with a minimum grout space area of 5400 mm^2 for filled grout spaces.

(b) Where the wall has been designed for elastic response to seismic loads and the grout spaces filled by the low lift procedure, the grout space shall not be less than that specified in NZS 4210.

C7.3.2

Where walls are constructed using the high lift grouting method, it is essential that the flue size be adequate to allow the grout to completely fill all the cavities to be grouted. For horizontal flues in reinforced hollow masonry the flue area may be taken as the height of the unit times the flue width, rather than the minimum flue area, which will occur above a depressed web.

Where the wall is constructed using the low lift grouting method, and provided that lifts terminate at every bond beam, the less stringent flue sizes specified by NZS 4210 may be adopted. However, it should be realized that high lift grouting with reduced composition is not considered a satisfactory construction method for walls designed for ductile seismic response.

Note that information pertaining to high lift grouting and to low lift grouting is detailed in NZS 4210.

7.3.3 *Dimensional limitations*

The thickness of structural walls shall be not less than 140 mm or 0.05 L_n whichever is the greater.

C7.3.3

This clause relates to the vertical load carrying capacity and stability of walls. For a minimum wall thickness of 140 mm and a maximum clear vertical distance of not greater than 2800 mm between floors, no vertical wall stiffenings will be required. Similarly, for a 190 mm wall thickness a maximum vertical spacing of 3800 mm is acceptable without requiring vertical wall stiffenings. Consequently, no vertical wall stiffeners will be required for normal storey heights when using 190 mm wall thickness. For a given wall thickness, b, and the case when lines of horizontal support have a clear vertical spacing of greater than b/0.05, then vertical lines of support having a clear horizontal spacing of not greater than b/0.05 shall be provided. These provisions should not be confused as being the limiting factor for the spacing of pilasters in non-loadbearing walls in single storey factory buildings.

7.3.4 *Wall reinforcement*

7.3.4.1 *Reinforcement requirements*

All reinforcement shall comply with the requirements of section 6.

7.3.4.2 *Distribution of wall reinforcement*

Wall reinforcement shall be distributed as follows:

(a) Horizontal reinforcement shall be uniformly distributed throughout the height of the wall except as otherwise allowed by 7.3.4.7;

(b) At least 50 % of the required vertical reinforcement shall be uniformly distributed along the length of the wall, subject to the minimum reinforcement provisions of 7.3.4.3, which shall be uniformly distributed.

7.3.4.3 *Minimum reinforcement*

All walls shall be reinforced both vertically and horizontally. The horizontal reinforcement shall be uniformly distributed up the wall height, except as allowed by 7.3.4.4 or 7.3.4.7.

Except as allowed by 7.3.4.4, the minimum area of reinforcement in each direction shall be 0.07 % of the gross cross-sectional area of the wall taken perpendicular to the orientation of the reinforcement considered. Minimum reinforcement shall comply with the following:

(a) Running bond

The sum of the horizontal and vertical reinforcement ratios shall be at least 0.2 % of the gross cross-sectional area in all cases.

(b) Stack bond

For stack bonded walls the minimum horizontal reinforcement ratio shall be:

(i) For building importance level 1 .... 0.07 %
(ii) For building importance level 2 or 3 .... 0.14 %
(iii) For building importance level 4 or 5 .... 0.25 %

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where the building importance level is determined by AS/NZS 1170.0.

C7.3.4.3

Although the total quantity of reinforcement is specified and a difference in distribution between horizontal and vertical reinforcement is allowed, wherever possible the distribution should be *approximately equal in each direction. The variation allowed is to accommodate limitations set by the range of available bar sizes and flue spacing in reinforced masonry.*

The performance in earthquakes of stack bonded walls without closely spaced horizontal reinforcement has been much less satisfactory than for walls constructed in running bond. The specified increase in minimum horizontal reinforcement for such walls is to ensure that the 'vertical columns' of masonry are well tied together. The minimum percentage of vertical reinforcement still applies.

7.3.4.4 *Arrangement of minimum reinforcement in short walls*

For walls less than 600 mm in length constructed in running bond the total reinforcement quantity of 0.2 % required by 7.3.4.3(a) may be placed vertically provided horizontal reinforcement is not required to carry shear.

C7.3.4.4

The intention of this clause is to allow small pilasters between openings in walls to be detailed without horizontal reinforcement, provided shear stresses are low. To satisfy this requirement, the pier must remain elastic under seismic loading, and be subjected to a maximum shear stress not greater than the grade-dependent value of v_m given in table 10.1.

7.3.4.5 *Maximum diameter of reinforcement*

The diameter of bars used in walls shall not exceed:

(a) One quarter of the least dimension of the grout space or cavity containing reinforcement; nor

(b) One eighth of the gross wall thickness.

C7.3.4.5

One of the major points of concern with reinforced masonry is that of achieving a construction that will allow an adequate vibrator or rodding space. This clause and the following clause are intended to set practical upper limits for reinforcement in flues and cavities, and should prevent undue congestion. It is recommended that wherever possible there should be one bar only in each flue, except at laps, particularly where the flue or cavity size approaches the lower limits for clear flue size.

This requirement limits the maximum bar diameter in a 140 mm wall to 16 mm, and in a 190 mm wall to 20 mm.

7.3.4.6 *Maximum reinforcement*

The maximum area of reinforcement in a grout space or cavity shall not exceed:

$$\frac{8}{f_y} \times \text{ the area of the grout space or cavity,}$$

except that at laps the total area of reinforcement may be:

$$\frac{13}{f_y} \times \text{ the areas of a grout space or cavity.}$$

C7.3.4.6

For 190 mm concrete masonry, with typical vertical flue dimensions of approximately 120 mm x 150 mm (18 000 mm^2), the maximum reinforcement area is 480 mm^2 for reinforcement having f_y = 300 MPa, or 288 mm^2 for reinforcement having f_y = 500 MPa; for example, one D20, two D16, or one DH16 bar per flue. At laps, the corresponding areas of 780 mm^2 and 468 mm^2 for f_y = 300 MPa and f_y = 500 MPa respectively would permit the lapping of one D20 bar per cell, the staggered lapping of one bar when having two D16 bars per cell, or the lapping of one DH16 bar in one cell, but not the lapping of two D16 bars or a DH20 bar. For 140 mm concrete masonry with a flue area of approximately 12 000 mm^2, one D16, two D12, or one DH12 bar could be lapped in a flue space.

7.3.4.7 *Horizontal reinforcement*

Horizontal reinforcement need not be uniformly distributed when all the following requirements are met:

(a) The building is located in an area for which the seismic hazard factor, Z, is less than 0.26; and

(b) Building height to the top of structural walls shall not exceed 7 m; and

(c) Masonry is built in running bond; and

(d) The wall has been designed for elastic seismic loading corresponding to $\mu = 1.0$, $S_p = 1.0$; and

(e) The design shear stress does not exceed the type-dependent value, v_m, given in 10.3.2.5.

7.3.4.8 *Empirical design for axial load strength*

Gravity loadbearing walls that are not required to respond in a limited ductile or ductile manner, may be designed by the empirical provisions of Eq. 7-1 if the resultant axial load at the ultimate limit state is located within the middle third of the overall thickness of the wall.

Nominal axial load strength, N_{nw}, of a wall shall be given by:

$$N_{nw} = 0.5\, f'_m\, A_g\, [1 - (L_n/40b)^2] \qquad \text{(Eq. 7-1)}$$

7.4 Principles and requirements additional to 7.3 for structures designed using a limited ductile or ductile seismic design philosophy

7.4.1 *Design compatibility*

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In the design of earthquake resisting ductile walls subjected to seismic actions at the ultimate limit state, the requirements of 3.7.3 for limited ductile walls and 3.7.4 for ductile walls shall be satisfied.

7.4.2 *Consideration of interaction between webs and flanges*

Cantilever or coupled shear walls shall be considered as integral units. The strength of flanges, boundary components, and webs shall be evaluated on the basis of compatible interaction between these elements using special study.

7.4.3 *Assumed extent of potential plastic hinge region*

The vertical extent of the potential plastic hinge region from the critical section in the wall shall be the greatest of:

(a) The length of the wall in the plane of forces resisting the seismic loads;

(b) One sixth the clear height of the wall.

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7.4.4 *Dimensional limitations*

7.4.4.1 *Minimum thickness in wall plastic hinge regions*

In potential plastic hinge regions of limited ductile or ductile structures the thickness of any part of structural walls, three storeys or higher, located in the outer half of the neutral axis depth when accounting for compression strains generated by the combination of axial load and flexure due to design load, shall be not less than 0.075 L_n unless:

(a) $c \leq 4b$; or

(b) $c \leq 0.3\, L_w$; or

(c) $c \leq 6b$ from the inside of a wall return of minimum length 0.2 L_n.

➤

C7.4.4.1

A thin wall subjected to high compression forces in the flexural compression zone is in danger, under cyclic loading, of premature failure by instability. Any such failure is unacceptable and must be guarded against.

It is considered unlikely that failure due to lateral instability of the wall will occur in structures less than 3 storeys high, because of the rapid reduction in flexural compression force with height, and for such walls the minimum wall thickness of 0.05 L_n *governs as detailed in 7.3.3.*

For limited ductile and ductile walls having a height of three storeys or greater the wall thickness within the outer portion of the compression zone is required to be at least 0.075 L_n*. Alternatively the designer can check the depth of the neutral axis for compliance with any one of the options given which may allow the lesser thickness limit to be applied.*

The thickness limitation of 0.075 L_n *implies a maximum unsupported wall height of 1870 mm for 140 mm thick walls, and 2530 mm for 190 mm thick walls. It should be noted that increasing the design value of* f'_m *above 12 MPa (implying special testing for masonry compression strength,* f'_m*) may result in sufficient reduction in the extent of the compression zone to allow the less onerous requirement of* $b = 0.05\ L_n$ *to apply.*

7.4.4.2 *Minimum thickness of parts of structural walls not within plastic hinge regions*
Outside potential plastic hinge regions the thickness of structural walls shall be not less than 140 mm or 0.05 L_n whichever is the greater.

7.4.4.3 *Minimum length of wall plastic hinge regions*
The length of a structural wall within the potential plastic hinge region shall be not less than $L_w = 790$ mm.

7.4.5 *Wall reinforcement*

7.4.5.1 *Vertical reinforcement in wall plastic hinge regions*
Within the potential plastic hinge region of structural walls the following requirements shall apply:

(a) Minimum vertical reinforcement size shall be D12;

(b) Maximum spacing of vertical reinforcement shall be 400 mm;

(c) Minimum number of vertical bars shall be 4;

(d) All vertical reinforcement shall be distributed uniformly along the length of the wall.

C7.4.5.1

Where reinforcement ratios and axial load levels are low, as will generally be the case for masonry walls, there is no advantage in concentrating flexural reinforcement at wall ends. Walls with equal quantities of flexural reinforcement will have essentially the same moment capacity regardless of whether this reinforcement is uniformly distributed or concentrated in two groups, one at each end of the wall. The following are strong reasons for avoiding high concentrations of reinforcement at wall ends:

(a) High concentration makes grouting and vibration difficult in crucial end compression regions;

(b) When crushing of potential plastic hinge regions occurs under extreme seismic attack, buckling of end compression bars will result. This effect would cause rapid degradation of wall strength if half the total wall reinforcement was so affected;

(c) Shear transfer and dowel action are improved by distributed reinforcement;

(d) A proportion of the total reinforcement will need to be distributed to satisfy minimum requirements.

7.4.5.2 *Horizontal reinforcement in wall plastic hinge regions*

Within the potential plastic hinge region of structural walls, the following requirements shall apply:

(a) Maximum spacing of horizontal reinforcement in walls not greater than 3 storeys or 12 m in height shall be 400 mm;

(b) Maximum spacing of horizontal reinforcement in walls greater than 3 storeys or 12 m in height shall be 200 mm;

(c) Horizontal reinforcement shall not be lapped within either 600 mm or 0.2 L_w, whichever is greater, from the end of the wall.

C7.4.5.2

The provisions of this clause are intended to provide adequate 'basketing' of masonry within the potential plastic hinge region. Tall walls will generally have higher ductility demand than shorter walls, and the degree of damage within the potential plastic hinge region can therefore be expected to be greater. Consequently closer spacing of horizontal reinforcement is required for walls higher than 3 storeys or 12 m. The maximum spacing requirements of $L_w/4$ as defined in 10.3.3.5 will govern for walls less than 3 storeys or 12 m high, and less than 1.6 m long.

7.4.5.3 *Reinforcement outside wall plastic hinge regions*

Outside the potential plastic hinge regions of structural walls the required reinforcement shall comply with 7.3.

7.4.5.4 *Lap splices in wall plastic hinge regions*

Lap splices may be used in potential plastic hinge regions of walls being part of structures designed using a limited ductile or ductile design philosophy, provided the lap length is not less than 60 d_b for reinforcement having f_y = 300 MPa or 105 d_b for reinforcement having f_y = 500 MPa, and provided that where h_w/L_w is greater than unity, not more than 50 % of reinforcement is to be lapped at any one level.

C7.4.5.4

Lapping of shear reinforcement within potential plastic hinge regions should be avoided wherever possible, and must not occur near the ends of the wall where crushing and vertical cracking of the masonry may occur under seismic loading, creating poor bond conditions for the horizontal reinforcement.

7.4.6 *Ductility considerations*

7.4.6.1 *Maximum neutral axis depth in potential plastic hinge regions*
Except as provided by 7.4.6.4 or 7.4.6.6, cantilever walls with aspect ratio $h_w/L_w \le 3$ shall be designed such that the neutral axis depth, within the potential plastic hinge region, under the most adverse loading condition complies with the following:

(a) For limited ductile walls:

  (i) For cantilever walls: $c \le 0.2\ L_w$;

  (ii) For walls with a contraflexure point between adjacent heights of lateral support: $c \le 0.45\ L_w^2/\ L_n$.

(b) For ductile walls:

$$c \le 0.4\ L_w/\mu \qquad \text{(Eq. 7-2)}$$

unless adequate ductility is confirmed by a special study as provided for in 7.4.6.2.

C7.4.6.1
To avoid failure of potential plastic hinge regions of unconfined masonry shear walls, it is necessary to limit the extreme fibre compression strain at the full design inelastic response displacement to the unconfined ultimate compression strain of $\varepsilon_u = 0.003$. The available ductility at this ultimate compression strain decreases with increasing depth of the compression zone, expressed as a fraction of the wall length. Equation 7-2 ensures that the available ductility will exceed the structural ductility factor, μ, for walls of aspect ratio less than 3. The lower the aspect ratio, the more conservative will be the results given by Equation 7-2. Less conservative results may be obtained from rational analyses.

Amd 1 Dec '06 *Where design flexural strength is in excess of that required for the structural ductility factor μ, the maximum neutral axis depth in Equation 7-2 may be increased in proportion to the excess strength provided.*

7.4.6.2 *Confirmation of available ductility*
For cantilever walls with aspect ratio $h_w/L_w > 3$, or for wall frames where stiffness of the connecting elements is such as to significantly modify basic cantilever action in the walls, a special study shall be carried out to ensure that the available displacement ductility, measured at the top of the wall or wall frame attains the required structural ductility. For the purpose of assessing section curvatures, the effective plastic hinge length shall be found from special study or taken as the smaller of half the wall length, or $0.2M^*/V$, but need not be taken less than a quarter of the wall length. Amd 1 Dec '06

C7.4.6.2
Where the available ductility is less than the adopted structural ductility factor μ redesign will be necessary. Options available to the designer are the use of a higher masonry compression strength, or wider or flanged walls. A further alternative is the inclusion of confining plates in critical regions of mortar beds within the potential plastic hinge region, which has been shown[7.1] *to increase the available ductility by a factor of at least three.*

7.4.6.3 *Ultimate compression strain for unconfined masonry*
For unconfined masonry, the available ductility shall be based on an ultimate compression strain of $\varepsilon_u = 0.003$.

7.4.6.4 *Ultimate compression strain for confined masonry*
Where confining plates in accordance with 7.4.6.5 are placed in critical mortar beds within the potential plastic hinge region, as defined in 7.4.3, available ductility shall be based on an ultimate compression strain of $\varepsilon_u = 0.008$. Amd 1 Dec '06

7.4.6.5 *Requirements for confining plates*

Confining plates placed in potential plastic hinge regions as shown in figure 7.1 to increase ductility shall satisfy all the following requirements:

(a) Be constructed of stainless steel or galvanized steel to the durability requirements of this Standard;

(b) Have minimum effective area of confining plate in each horizontal direction cut by a vertical section of area, $s_h h''$, of:

$$A_p \geq 0.004\ s_h h'' \quad \text{(Eq. 7-3);}$$

(c) Provide confinement from the critical section for a height not less than the extent of the potential plastic hinge region defined by 7.4.3 and for a horizontal distance not less than 0.625*c* or 600 mm, whichever is greater, from the extreme compression fibre;

Amd 1 Dec. '06 (d) Have vertical spacing within the potential plastic hinge region not exceeding $s_h = 200$ mm.

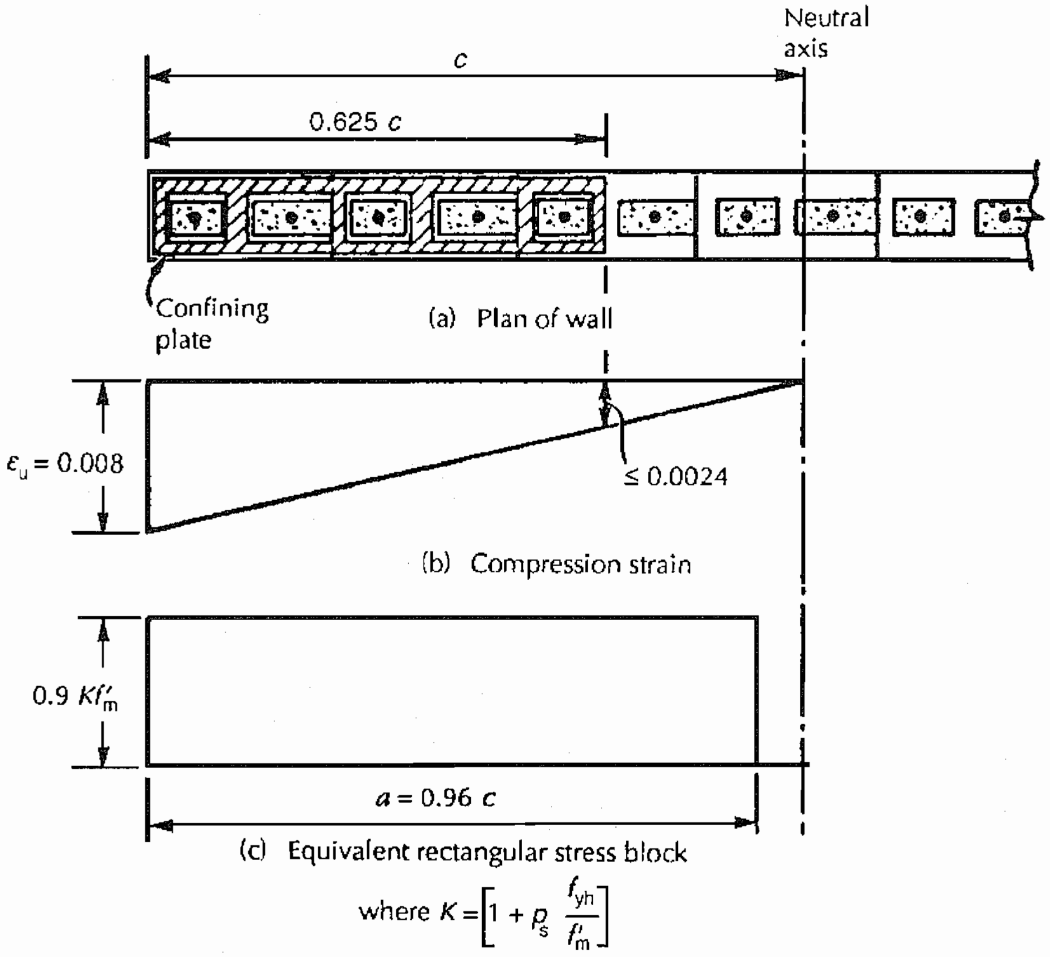


Figure 7.1 – Confining plates in potential plastic hinge regions

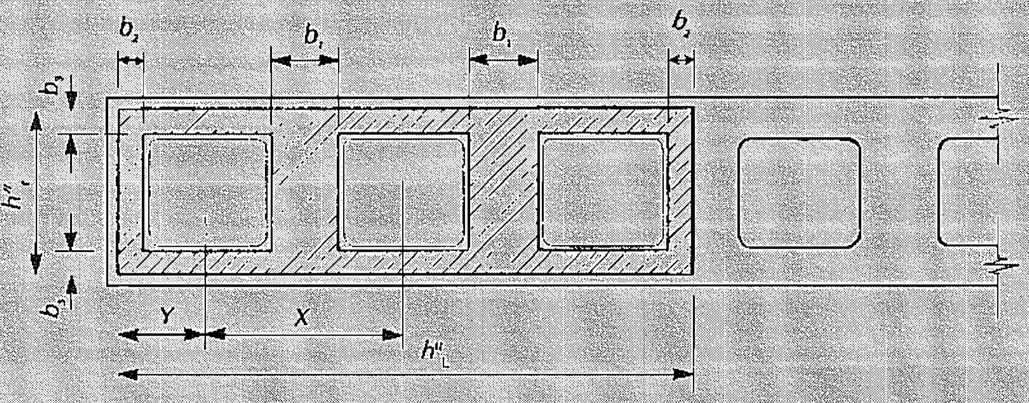
C7.4.6.5

Where confining plates are used, they must be continuous over masonry unit header joints for the full required length. The required plate area transverse to the wall should be obtained by cross links above each masonry unit web, and should not significantly affect grout placing or vibrating. Figures 7.1 and 7.2 illustrate the requirements of 7.4.6.5. Covering the most highly strained 62.5 % of the compression zone ensures that no unconfined fibres are subjected to strains in excess of 0.003. The minimum length of 600 mm is intended to ensure that the end two vertical flues, and any reinforcement they contain, are tied back into the body of the wall, minimizing the influence of weak header joints close to the extreme compression fibre. The vertical extent of confinement should correspond to the height of the potential plastic hinge region.

When using confinement plates the shape of the equivalent rectangular stress block for ultimate flexural strength calculation is as shown in figure 7.1(c). This stress block and the ultimate strain of $\varepsilon_u = 0.008$ *should be used when assessing available ductility of confined masonry walls. Alternatively, the ductility charts for confined rectangular masonry walls, given in Reference 7.2, may be used.*

Where confining plates are used it is necessary to check their cross-sectional areas in each direction. Refer to figure 7.2.

The width of the individual cross webs of the confining plate should be approximately proportional to the contributing confinement length. Refer to figure 7.2.



$A_{p1} = 2tb_3$, or

$A_{p2} = 2t(b_1 + b_2)$ – for this example only.

Amd 1 Dec '06 NOTE – Width of cross webs should be approximately proportional to the contributing confinement length, i.e.

$$\frac{X}{b_1} \approx \frac{Y}{b_2}$$

p_s shall be the smaller of: $\dfrac{A_{p1}}{s_h . h''_t}$ or $\dfrac{A_{p2}}{s_h . h''_L}$

Figure 7.2 – Confining plate example showing h'', A_p and p_s

7.4.6.6 *Confinement to NZS 3101 where two layers of reinforcement are provided*

Where two parallel layers of vertical reinforcement are provided in the compression zone of potential plastic hinge regions, rectangular hoops or ties shall be used to provide confinement to the requirements of NZS 3101.

C7.4.6.6
In masonry walls of sufficient width to enable two layers of vertical reinforcement to be placed one adjacent to each face, it would be possible to provide ductility by using confining hoops or ties within the grout space, in accordance with provisions for confined walls in NZS 3101. However, because of the high cover inherent in masonry walls this will inevitably result in high areas of transverse reinforcement being required, and will rarely lead to an economic design. In such cases the concrete masonry mix should be considered as permanent formwork only with all loads assumed to be carried only by the grout core, enabling use of the specified compression strength of the grout (NZS 4210 specifies a minimum of 17.5 MPa) compared with the values of masonry as specified in table 3.1.

7.4.7 *Shear strength*

7.4.7.1 *General*
The evaluation of wall shear strength shall be in accordance with 10.3.

7.4.7.2 *Confinement reinforcement assumed to contribute to shear strength*
Hoops or ties, where provided for confinement, may be assumed to contribute to the shear strength of a wall element.

7.4.8 *Walls with openings*

7.4.8.1 *Arrange openings to avoid unintentional failure planes*
Openings in structural walls shall be so arranged that unintentional failure planes across adjacent openings do not reduce the shear or flexural strength of the structure. For ductile cantilever walls with irregular openings, appropriate analyses such as based on strut-and-tie models shall establish rational paths of the internal forces. Capacity design procedures shall be used to ensure that the horizontal shear reinforcement will not yield before the flexural strength of the wall is developed.

C7.4.8.1
The structural significance of openings in walls must be carefully considered to ensure strength and ductility capacity do not differ from the designer's assumptions. Larger openings result in structural modification to the coupled shear wall behaviour which requires substantial ductility of the coupling beam units. Further details are provided in References 7.2 and 7.3.

7.4.8.2 *Requirements for coupling beams in ductile coupled shear walls*
Walls or elements of ductile coupled shear walls shall be connected by ductile coupling beams. Such beams shall comply with the following requirements:

(a) The compression zone depth in a coupling beam at nominal flexural strength shall not exceed:

$$c = \frac{1.2\,h_b^2}{\mu L_n} \quad \text{(Eq. 7-4);}$$

(b) The total shear stress, v_n , in coupling beams shall not exceed v_g or the value:

$$v_n = \left(\frac{L_n}{\mu h_b}\right) v_g \quad \text{(Eq. 7-5)}$$

where v_g is given in 10.3.2.4;

➤

(c) Coupling beams shall contain horizontal and vertical reinforcement not smaller than D12 bars at not greater than 200 mm centres each way;

(d) Shear design of coupling beams and walls shall be designed to capacity design principles;

(e) Minimum depth of coupling beams shall be not less than 800 mm;

(f) Minimum width of coupling beams shall be not less than 190 mm.

C7.4.8.2
As with NZS 3101, coupled shear walls are treated as a special case of walls with openings. However, the requirements of 7.4.8.2 are intended to apply to the full range of structural systems consisting of walls, with short, stiff spandrel beams to wall frames, where the beams may be much more flexible.

Design of coupled shear walls[7.3] *should be based on accepted capacity design principles to ensure that flexural hinging can occur only to the coupling beams, and at the base of the walls. Nominal shear strength of coupling beams and walls should exceed that corresponding to flexural overstrength developing in all potential plastic hinges.*

It should be noted that stack bonding of masonry units is not permitted in potential plastic hinge regions and therefore cannot be used for coupling beams. The use of stack bonded deep lintel concrete units (190 mm long x 390 mm high) is thus not permitted unless the coupling beams are designed for elastic response under earthquake loading.

7.4.8.3 *Confirm available ductility of coupling beams*
Ductile coupled masonry shear walls exceeding four storeys in height shall be the subject of a special study to confirm that the available ductility of all coupling beams and walls exceed that implied by the adopted structural ductility factor μ.

REFERENCES

7.1 Priestley, M.J.N., "Ductility of Confined Masonry Shear Walls", Bull. NZ National Society for Earthquake Engineering, Vol. 15, No. 1, March 1982, pp. 22-26.

7.2 Ingham, J.M., and Voon, K.C., "NZS 4230:2004 User Guide", Section 4.1, New Zealand Concrete Masonry Manual, 2004.

7.3 Paulay, T., and Priestley, M.J.N., "Seismic design of Reinforced Concrete and Masonry Buildings", John Wiley and Sons, 1992.

8 BEAMS

8.1 Notation

A_{tr}	Smaller of area of transverse reinforcement within a spacing, s, crossing plane of splitting normal to masonry surface containing extreme tension fibres, or a total area of transverse reinforcement normal to the layer of bars within a spacing, s, divided by n, mm^2
b_w	Effective web width, mm
c_{max}	Maximum neutral axis depth, mm
d	Distance from extreme compression fibre to centroid of tension reinforcement, mm
d_b	Nominal diameter of bar, wire, or in a bundle, the diameter of a bar of equilvalent area, mm
f'_m	Specified compressive strength of masonry, MPa
f_y	Lower characteristic yield strength of non-prestressed reinforcement, MPa
f_{yt}	Lower characteristic yield strength of transverse reinforcement, MPa
h	Overall depth of component in the plane of loading, mm
L_d	Development length, mm
L_n	Clear vertical distance between lines of effective horizontal support or clear horizontal distance between lines of effective vertical support, mm
M_n	Nominal flexural strength of section, Nm
n	Number of bars in a layer
p_{max}	Maximum ratio of tension reinforcement
p_{min}	Minimum ratio of tension reinforcement
s	Maximum spacing of transverse reinforcement within L_d, or spacing of stirrups or ties or spacing of successive turns of a spiral, all measured centre-to-centre, mm
V^*	Design shear force at section at ultimate limit state, N
ϕ	Strength reduction factor, specified in 3.4.7
μ	Structural ductility factor, specified in AS/NZS 1170

C8.1

The following symbols which appear in this clause are additional to those used in section 8.

A_m	*Usable area of masonry section, mm^2*
c	*Neutral axis depth from extreme compression fibre, mm*
h_u	*Height of masonry unit, mm*

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N^* *Design axial load in compression at given eccentricity, N*

p_b *Ratio of tension reinforcement at balanced strain conditions*

t_c *Width of cell in masonry unit, mm*

8.2 Scope

The design of all beams, irrespective of the seismic design philosophy, shall comply with 8.3. Clause 8.4 provides requirements additional to 8.3 for structures that are designed assuming limited ductile or ductile behaviour in an earthquake.

8.3 General principles and requirements for all structures

8.3.1 *General design principles*

8.3.1.1 *Design loads*

Beams shall be designed for all vertical, lateral in-plane and face loadings to which they may be subjected including provision for eccentric loads.

8.3.1.2 *Design for flexure with or without axial loads*

Beams subjected to combined flexure and axial loads shall be designed in accordance with the principles of 10.2 including the effects of slenderness except as modified by the provisions of 8.3.1.

C8.3.1.2

Beams will typically be horizontal, or near horizontal, in elevation and carry loads which are at right angles to their centroidal axis. When a beam is subjected to horizontal loads which are parallel to its centroidal axis, and the beam is not tied into a concrete diaphragm, then a check should be made on the maximum expected level of axial load in the beam. If the design axial load N exceeds* $0.1\phi f'_m A_m$ *then the beam should be designed as a column or wall.*

Although beams in reinforced concrete or steel may be expected to be slender horizontal components, masonry beams are often part of a structural wall system where the beam depth to span ratios are more likely to be around 1 to 4, rather than 1 to 10 as for concrete. Such components are still designed primarily for flexure and come within the scope of this section of the Standard. Provisions are included, to ensure that these rather deep masonry beams have reinforcement well distributed throughout their depth and length.

8.3.1.3 *Design for shear and torsion*

Beams subjected to shear and/or torsion shall be designed to the provisions of 10.3 except as modified by the provisions of 8.3.1.

8.3.1.4 *Beam-column joints*

Elements that form part of a beam-column joint shall be designed in accordance with section 11.

8.3.2 *Components subjected to pure flexure*

Components subjected to pure flexure shall be designed as beams.

8.3.3 *Beam construction*

8.3.3.1 *Grouting beams*

All masonry beams shall be completely grouted.

8.3.3.2 *Minimum grout space dimensions*
Beams shall be constructed so that the grout spaces have a minimum clear dimension of 60 mm and a minimum area of 5 400 mm^2.

8.3.4 *Dimensional limitations for beams*
When designing beams the following minimum dimensions apply:

(a) The minimum width of a beam or web of a flanged T- or L-beam shall not be less than 140 mm;

(b) The minimum depth of beams in the vertical direction for beams not supporting or attached to partitions or other construction likely to be damaged by large deflections, shall be not less than the following:

Simply supported	0.1 L_n
One end continuous	0.083 L_n
Both ends continuous	0.071 L_n
Cantilever	0.2 L_n
Width of beam subject to horizontal load	0.042 L_n

Variations from these figures shall be the subject of a special study.

***C8.3.4** Dimensional limitations for beams*
Although this clause allows the construction of shallow beams, the specific requirements for minimum flue sizes and lateral restraint for the beam still apply.

8.3.5 *T-beams*

8.3.5.1 *Effective widths for T-beam*
In T-beam construction, the flange and web shall be built integrally unless shown by a special study to be otherwise effectively bonded together, and the following shall apply;

(a) The effective width of a flange resisting action due to flexure shall not exceed one quarter of the span length of the beam or one third of the height of the shear wall, whichever is appropriate, and the effective overhanging flange width on each side of the web shall not exceed:

   (i) Eight times the flange thickness; nor

   (ii) Half the clear distance to the next web.

(b) For beams or shear walls with a flange on one side only, the effective overhanging slab width considered in flexural resistance shall not exceed:

   (i) One twelfth the span length of the beam, or one ninth of the height of the shear wall, whichever is appropriate; nor

   (ii) Six times the slab thickness; nor

   (iii) Half the clear distance to the next web.

➤

(c) Isolated beams, in which the T-shape is used to provide a flange for additional compression area, shall have a flange thickness not less than one half the width of web and an effective flange width not more than four times the width of web. In such beams transverse reinforcement placed perpendicularly to the beam shall be provided so as to:

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   (i) Carry the factored actions on the overhanging slab width assumed to act as a cantilever;

   (ii) Act as shear reinforcement when necessary to ensure flange action;

   (iii) Be placed not further apart than five times the slab thickness, nor 450 mm.

8.3.5.2 *Moment of inertia for T-beams*

In computing the effective moment of inertia of cracked sections, the effective width of the overhanging parts of flanged components shall be one half of that given in 8.3.5.1.

8.3.6 *Longitudinal reinforcement in beams*

8.3.6.1 *Maximum diameter of longitudinal reinforcement*

The diameter of longitudinal reinforcement used in a beam shall not exceed:

(a) One quarter of the least dimension of the grout space or cavity containing reinforcement; nor

(b) One eighth of the least beam dimension.

C8.3.6.1

In the calculation of the maximum reinforcing area which can be contained in a grout space, the area of the grout space may be taken as $h_c \times t_c$, see figure 8.1.

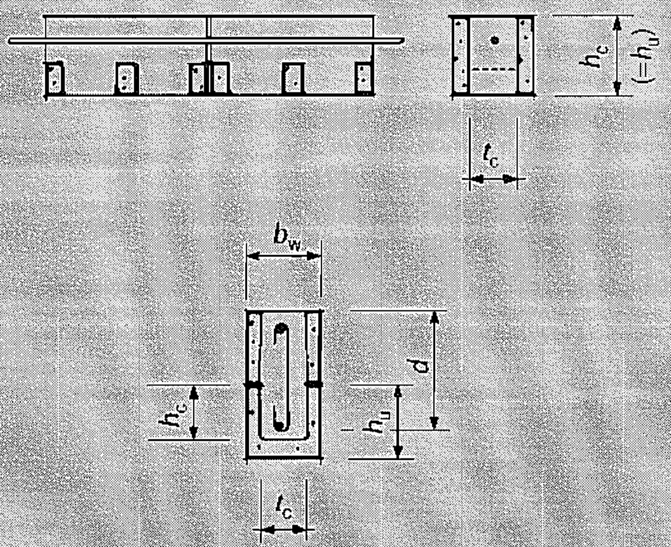


Figure 8.1 – Grout space dimensions

8.3.6.2 *Maximum area of reinforcement in grout space*

The maximum area of reinforcement in a grout space or cavity shall not exceed $8/f_y$ times the area of the grout space or cavity except that at laps the total area of reinforcement may be $13/f_y$ times the area of a grout space or cavity.

C8.3.6.2

One of the major points of concern with reinforced masonry is that of achieving a construction that will allow an adequate vibrator space. This clause and the following clause are intended to set practical *upper limits for reinforcement in flues and cavities, and should prevent undue congestion. It is recommended that wherever possible there should be one bar only in each flue, except at laps, particularly where the flue or cavity size approaches the lower limits for clear flue size.*

For 190 mm concrete masonry, with typical flue horizontal dimensions of approximately 120 mm x 150 mm (18 000 mm^2), the maximum reinforcement area is 480 mm^2 for reinforcement having f_y = 300 MPa, or 288 mm^2 for reinforcement having f_y = 500 MPa; for example, one D20, two D16, or one DH16 bar per flue. At laps, the corresponding areas of 780 mm^2 and 468 mm^2 for f_y = 300 MPa and f_y = 500 MPa respectively would permit the lapping of one D20 bar per cell, the staggered lapping of one bar when having two D16 bars per cell, or the lapping of one DH16 bar in one cell, but not the lapping of two D16 bars or a DH20 bar. For 140 mm concrete masonry with a flue area of approximately 12 000 mm^2, one D16, two D12, or one DH12 bar could be lapped in a flue space.

8.3.6.3 *Minimum tension reinforcement ratio*

The following minimum tension reinforcement ratios shall apply:

(a) Except as detailed in 8.3.6.3 (b) and 8.3.6.4, at any positive moment section of a beam where tension reinforcement is required by analysis, the ratio calculated using the width of web for a T-beam, shall not be less than that given by:

$p_{min} = 0.7 / f_y$ .... (Eq. 8-1)

(b) Alternatively, the area of reinforcement provided at every section of a beam for positive and negative moment, shall be at least one third greater than that required by analysis but in no case shall p_{min} be less than 0.07 %.

C8.3.6.3

The minimum reinforcement percentages given are half of those in NZS 3101. These figures take into consideration the lower design stresses, the lower cracking strength of masonry construction, and the fact that the reinforcement may be distributed through the depth of the flexural tension zone rather than concentrated close to the extreme tension fibre.

8.3.6.4 *Maximum tension reinforcement ratio*

The ratio of longitudinal reinforcement provided in beams shall not exceed 0.75 of the ratio that would produce balanced strain conditions for the section under flexure without axial load.

C8.3.6.4

Although this clause sets upper limits for the maximum area of reinforcement based on 0.75 p_b, it does not limit the total area of reinforcement which can be located within a beam grout space, which must still satisfy 8.3.6.2. Table 8.1 gives values for the maximum reinforcement in a typical masonry lintel beam, assuming d = 290 mm, t_c = 120 mm and h_c = 150 mm.

Table 8.1 – Longitudinal reinforcement limits in a 390 mm x 190 mm masonry lintel beam

Observation type	**Design f'_m** (MPa)	**Reinforcement f_y** (MPa)	**Max. area of reinforcement $0.75p_b$** (mm^2)	**Max. area of reinforcement $8\,t_c\,h_u / f_y$** (mm^2)	**Min. area of reinforcement $0.7\,b_w d / f_y$** (mm^2)
Observation Type C	4	300	312	480	129
		500	153	288	78
Observation Type A + B	12	300	936	480	129
		500	459	288	78

8.3.7 *Development of positive moment reinforcement in beams*

8.3.7.1 *Quantity of reinforcement to extend to support*

At least 33 % of the maximum positive moment reinforcement in simple components and 25 % of the maximum positive moment reinforcement in continuous components shall extend along the same face of the component into the support. In beams, such reinforcement shall extend into the support by at least 200 mm.

8.3.7.2 *Anchorage at supports of primary lateral load resisting system*

When a flexural component is part of a primary lateral load resisting system, positive moment reinforcement required to be extended into the support by 8.3.7.1 shall be anchored to develop the lower characteristic yield strength, f_y, in tension at the face of support. Anchorage of longitudinal beam reinforcement into the column core is detailed in 8.4.4.2.

8.3.7.3 *Limitation of flexural bond stress at supports*

The positive tension reinforcement at simple supports shall be limited in diameter to enable the bars extending to the free end of the component to be fully developed from a point M_n/V^* from the centre of the support. The value of M_n/V^* may be increased by 30 % when the ends of reinforcement at the supports are confined by a compressive reaction.

8.3.7.4 *Limitation of flexural bond stress at points of inflection*

The positive and negative tension reinforcement at points of inflection shall be limited in diameter to enable bars, from a point M_n/V^* from the point of inflection to be fully developed satisfying the requirements that:

$$L_d \leq \frac{M_n}{V^*} + 12\, d_b \quad \text{(Eq. 8-2)}$$

The value of M_n/V^* may be increased by 30 % when the ends of reinforcement are confined by a compressive reaction.

8.3.8 *Development of negative moment reinforcement in beams*

8.3.8.1 *General*

Negative moment reinforcement in a continuous restrained or cantilever component, or in any component of a rigid frame, shall be anchored in or through the supporting component by embedment length, hooks or mechanical anchorage.

8.3.8.2 *Development of negative reinforcement*

Negative moment reinforcement shall have an embedment length into the span as required by 6.3.7 and 6.3.10.3.

8.3.8.3 *Curtailment of negative reinforcement*

At least one third of the total tension reinforcement provided for negative moment at a support shall have an embedment length beyond the point of inflection, according to the appropriate bending moment envelope, for a distance of not less than 1.3 times the effective depth of the component.

8.3.9 *Compression reinforcement in beams*

Compression reinforcement shall not be relied upon to enhance the strength of masonry beams, unless such reinforcement is supported against lateral buckling to the requirements of NZS 3101.

C8.3.9

To be effective, compression reinforcement should be restrained by ties in two directions. This restraint cannot be practically achieved in relatively narrow masonry beams. Until such time as practical means are devised for providing this restraint, the reliance on compression reinforcement to enhance beam flexural strength has been prohibited.

Where masonry beams require additional compression reinforcement, however, they may be redesigned as concrete beams to NZS 3101 using only the section dimensions and compressive strength of the grout in-fill to the beam.

8.3.10 *Distribution of flexural reinforcements in beams*

8.3.10.1 *Flexural reinforcement shall be evenly distributed*

In beams the flexural tension reinforcement shall be evenly distributed within the maximum flexural tension zones of a component cross-section.

8.3.10.2 *Reinforcement in grout spaces greater than 200 mm wide*

When the width of the grouted space or cavity exceeds 200 mm the beam shall have reinforcement adjacent to each vertical face at every horizontal layer of reinforcement used.

8.3.10.3 *Reinforcement of flanged beams*

Where the masonry beam has flanges in either masonry or concrete that is in tension, part of the flexural tension reinforcement may be distributed within an effective flange width as follows:

(a) For cantilevers – within the flange width defined in 8.3.5.1;

(b) For beams continuous over supports – within a flange width which does not exceed the width defined in 8.3.5.1 nor 0.1 L_n.

C8.3.10.3

The reference to beams with masonry flanges could also refer to cantilever pilaster-type construction, where the axial load on the wall and pilaster is nominal and the pilaster is designed for flexure and shear only. In this type of application, and also where a composite concrete and masonry beam cantilevers from a support, the reinforcement may be distributed over a greater flange width than where the beam spans between, and is continuous, over supports.

Although it is not a requirement that tension reinforcement shall be distributed in the flanges of T-beam type construction, it is a recommended practice which will assist in controlling cracking at service load.

Clause 8.4 has further specific requirements for ductile seismic design on the minimum reinforcement which is to be continuous through columns, and the reinforcement which is to be included in the calculation of the strength of beam sections with flanges.

8.3.10.4 *Distribution of reinforcement in beams greater than 800 mm deep*

If the overall depth of a beam exceeds 800 mm then longitudinal reinforcement with an area of 0.14 % shall be distributed in the flexural tension zone with a vertical spacing of not more than 400 mm. Where such reinforcement is included in strength computations a strain compatibility analysis shall be made to determine stresses in the individual bars.

C8.3.10.4

The minimum web reinforcement required to be distributed for crack control within the tension zone of the beam is required to be not less than minimum wall reinforcement. This reinforcement may be taken to contribute towards the flexural strength of the beam. Distributed reinforcement must be included in the calculation of flexural overstrength of sections for capacity design.

8.3.11 *Stirrup and tie reinforcement in beams*
All beams shall have minimum stirrup and tie reinforcement to the provisions of this section.

8.3.11.1 *Diameter limitations of stirrup and tie reinforcement*
Rectangular hoop or tie reinforcement for beams shall be at least 6 mm in diameter for longitudinal bars less than 20 mm in diameter, and 10 mm in diameter for longitudinal bars being 20 and 25 mm in diameter and for bundled longitudinal bars.

8.3.11.2 *Longitudinal reinforcement shall be enclosed*
Stirrup or tie reinforcement shall enclose the longitudinal reinforcement in beams.

8.3.11.3 *Spacing of stirrups and ties*
Centre to centre spacing and arrangement of stirrup or tie reinforcement shall not exceed the smaller of the least lateral dimensions of the cross-section of a component, $0.5h$, 16 longitudinal bar diameters or 48 bar diameters.

8.3.11.4 *Spacing of shear reinforcement*
Spacing of shear reinforcement, placed perpendicular to the axis of the component, shall not exceed $0.5h$ nor 600 mm for beams.

8.3.11.5 *Detailing requirements for stirrups and ties*
Stirrup and tie reinforcement shall be anchored by at least a 135° bend around a longitudinal bar plus an extension beyond the bend of at least 8 stirrup or tie bar diameters. Alternatively, the ends of the stirrup shall be spliced by welding to develop the breaking strength of the bar or $1.6\ f_y$, whichever is smaller.

8.3.12 *Distance between lateral supports of beams*

8.3.12.1
Spacing of lateral supports for a beam shall not exceed 30 times the least width of the beam.

C8.3.12.1
The limitation of 8.3.12.1 refers to the width of the beam only and designers are reminded in this clause that eccentric loads can become detrimental to narrow deep beams. Lateral supports closer than 30 times the beam width may be required by actual loading conditions. The spacing of lateral supports for masonry beams has been decreased from that given in NZS 3101 because of the lower flexural and torsional stiffness of masonry.

8.3.12.2
The effects of lateral eccentricity of load shall be taken into account in determining spacing of lateral supports.

8.3.13 *Deep beams*

8.3.13.1 *Definition of deep beam*
Beams with overall depth to clear span ratios greater than 0.4 for continuous spans, or 0.8 for simple spans, shall be designed as deep beams taking into account non-linear distribution of strain and lateral buckling.

8.3.13.2 *Shear design of deep beams*
The design of deep beams for shear effects shall be in accordance with 10.3.2.16.

8.3.13.3 *Minimum flexural reinforcement*
Minimum flexural reinforcement for deep beams shall conform to the requirements for beams.

8.3.13.4 *Minimum horizontal and vertical reinforcement*

The minimum horizontal and vertical reinforcement for deep beams shall be not less than the minimum reinforcement required for structural walls.

8.4 Principles and requirements additional to 8.3 for structures designed using a limited ductile or ductile seismic design philosophy

8.4.1 *Design to be compatible with principles in section 3*

Amd 1 Dec '06

In the design of earthquake resisting ductile beams subjected to seismic actions at the ultimate limit state, the requirements of 3.7.3 for limited ductile beams and 3.7.4 for ductile beams shall be satisfied.

8.4.2 *Dimensional limitations for beams*

C8.4.2

Limits have been imposed on the maximum dimensions of beams, to ensure that they behave in a ductile manner under seismic loading.

8.4.2.1 *Minimum width of compression face of beam*

The width of the compression face of a beam with rectangular T- or L-section shall be not less than 190 mm nor the value given by Equations 8-3 and 8-5.

8.4.2.2 *Minimum depth of beam*

The overall depth of a beam shall be not less than 390 mm nor the value given by Equations 8-4 and 8-6.

8.4.2.3 *Depth, width and clear span limitations for beams with moments at each end*

Depth, width, and clear length between the faces of supports of components with rectangular cross-section, to which moments are applied at both ends by adjacent beams, columns, or both, shall be such that:

$$L_n / b_w \leq 20 \qquad \text{(Eq. 8-3)}$$

and

$$L_n h / b_w^2 \leq 80 \qquad \text{(Eq. 8-4)}$$

8.4.2.4 *Depth, width and clear span limitations for cantilevered beams*

Depth, width, and clear length from the face of support of cantilever beams with rectangular cross-section, shall be such that:

$$L_n / b_w \leq 10 \qquad \text{(Eq. 8-5)}$$

and

$$L_n h / b_w^2 \leq 40 \qquad \text{(Eq. 8-6)}$$

8.4.2.5 *Width of T-beams*

The limiting values specified in Equations 8-3 and 8-5 may be increased by 50 % where the width of T- and L-beams comply with 8.3.5.1.

8.4.2.6 *Dimension of beam-column joints*

Dimensions of joints between beams and columns shall comply with the minimum dimensions set out in 11.4.2.

8.4.3 *Reinforcement in beams*

C8.4.3
The requirements of this clause are additional to the general requirements for beam longitudinal reinforcement which are given in 8.3.6.

8.4.3.1 *Extent of potential plastic hinge region*
Potential plastic hinge regions in beams shall be considered to be as follows:

(a) Where the critical section is located at the face of a supporting column, wall or beam:
Over a length equal to the beam depth, measured from the critical section towards midspan;

(b) Where the critical section is located at a distance not less than the beam depth, h, away from a column or wall face:
Over a length that commences between the column or wall face and the critical section, at least $0.5h$ from the critical section, and extends at least $0.5h$ past the critical section towards midspan.

C8.4.3.1
Masonry beams are often deeper and of shorter span length than concrete beams and therefore are less likely to be dominated by moments from gravity loads than concrete beams. Hinge areas within the beam can therefore be confidently expected to occur at or near the ends of the stiffer masonry beams than the more flexible and heavily loaded concrete beams. Hinge lengths have therefore been reduced from 2h in NZS 3101 to h in this Standard.

The separation of the potential plastic hinge away from the column face effectively increases the width of the beam-column joint and therefore a reduction is allowed in the minimum horizontal dimension of the joint (see 11.4.2.2). If, however, beam moments are gravity load dominated then designers should design to NZS 3101.

8.4.3.2 *Longitudinal reinforcement in potential plastic hinge region*
Within the potential plastic hinge region of a beam the following longitudinal reinforcement requirements shall apply:

(a) When the hinge region of the beam is not confined to the requirements of NZS 3101, the tension reinforcement shall be such that the depth from the extreme compression fibre to the neutral axis shall not exceed $c_{\max}$, where:

$$c_{\max} = \frac{1.2h^2}{\mu L_n} \quad \text{(Eq. 8-7);}$$

(b) The reinforcement ratio calculated using the width of the web shall not exceed:

$$p_{\max} = \frac{3.5}{f_y} \quad \text{(Eq. 8-8);}$$

(c) Minimum reinforcement shall consist of D12 bars at a maximum spacing of 200 mm centres;

(d) Principal tension reinforcement shall not be lapped or terminated;

(e) Beams with two or more bars in each layer shall have the bars restrained against buckling by ties in accordance with NZS 3101.

C8.4.3.2
Limits have been imposed on the maximum depth to neutral axis, c, to ensure that beams behave in a ductile manner under seismic loading. Lightly reinforced beams will have adequate curvature ductility without exceeding the maximum compression strain of 0.003. Equation 8-7 sets limits for this reinforcement. If, however, the hinge area is confined in accordance with NZS 3101 then the maximum allowable tension reinforcing is not limited by this clause but by either 0.75 p_b or 8.4.3.2(b).

Alternatively from the frame ductility displacement demand of μ the required available curvature ductility of the component could be calculated. If this curvature ductility was available at compression strains of 0.003 or less then special confinement reinforcing would not be required in the hinge area and the tension reinforcement limitation of 8.4.3.2(b) could be exceeded.

Principal reinforcement terminating within the potential plastic hinge region of beams increases the nominal moment capacity over that part of the hinge zone affected by the bar termination and reduces the length of the hinge region. This has the effect of reducing the hinge length and the overall ductility of the structure.

8.4.3.3 *Longitudinal reinforcement outside potential plastic hinge region*
Throughout the length of a beam, the following additional longitudinal reinforcement requirements shall apply:

(a) At least one D12 bar shall be provided both top and bottom throughout the length of the beam;

(b) At least one quarter of the top flexural reinforcement required at either end of a beam shall be continued throughout its length.

8.4.3.4 *Transverse reinforcement*
Transverse reinforcement inside and outside the potential plastic hinge region shall comply with the relevant requirements of NZS 3101.

8.4.3.5 *T-beams*
In T- and L-beams built integrally with slabs, the slab longitudinal reinforcement to be considered effective as beam reinforcement shall be determined in accordance with NZS 3101.

8.4.3.6 *Reinforcement bar diameter limitations*
Where beam bars pass through a joint, the beam bar diameter limitations shall comply with 11.4.2.2.

8.4.4 *Development of flexural reinforcement*

8.4.4.1 *Attainment of flexural overstrength in potential plastic hinges*
For structures designed using a limited ductile or ductile seismic design philosophy, the distribution and curtailment of the longitudinal flexural reinforcement shall be such that the flexural overstrength of a section can be attained at critical sections in potential plastic hinge regions.

8.4.4.2 *Anchorage into columns*
When longitudinal beam bars are anchored in column cores or beam stubs, the anchorage shall be deemed to commence:

(a) Where potential plastic hinge regions form in the beams at the column face, one-half of the relevant depth of the column or 10 d_b, whichever is less, from the face at which the beam bar enters the column; ➤

(b) Where the critical section of the hinge is at a distance of at least the beam depth or 500 mm, whichever is less, from the column face, the development length may be considered to commence at the column face of entry;

(c) For elastic or nominally ductile structures the development length may be considered to commence at the column face of entry.

8.4.4.3 *Confinement required at splices for beams*

Tensile reinforcement in beams shall not be spliced by lapping in a region where reversing stresses at the ultimate limit state may exceed 0.6 f_y in tension or compression unless each spliced bar is confined by stirrup ties so that:

$$\frac{A_{tr}}{s} > \frac{d_b}{48}\frac{f_y}{f_{yt}} \qquad \text{(Eq. 8-9)}$$

9 COLUMNS

9.1 Notation

A_m	Usable area of masonry section, mm^2
A_{st}	Total area of longitudinal reinforcement, mm^2
b_w	Effective web width, mm
c	Neutral axis depth from extreme compression fibre, mm
c_{max}	Maximum neutral axis depth, mm
EI	Flexural stiffness of a component
E_m	Modulus of elasticity of masonry, MPa
f'_m	Specified compressive strength of masonry, MPa
f_y	Lower characteristic yield strength of non-prestressed reinforcement, MPa
h	Overall depth of component in the plane of loading, mm
I_g	Moment of inertia of gross masonry section about the centroidal axis, neglecting reinforcement, mm^4
L_n	Clear vertical distance between lines of effective horizontal support or clear horizontal distance between lines of effective vertical support, mm
M^*	Design moment for a component, Nm
N^*	Design axial load in compression at given eccentricity, N
N^*_e	Design axial load in compression at given eccentricity due to gravity and seismic load acting on a component during an earthquake, N
N_o	Nominal axial compressive strength when the axial load is applied at zero eccentricity, N
β_d	Ratio of maximum design dead load moment to maximum design total load moment, always positive
ϕ	Strength reduction factor, specified in 3.4.7
μ	Structural ductility factor, specified in AS/NZS 1170

9.2 Scope

The design of all columns, irrespective of the seismic design philosophy, shall comply with 9.3. Clause 9.4 provides requirements additional to 9.3 for structures that are designed assuming limited ductile or ductile behaviour in an earthquake.

9.3 General principles and requirements for all structures

9.3.1 *General design principles*

9.3.1.1 *Design loads*
Columns shall be designed for all axial, lateral in-plane and face loadings to which they may be subjected including provision for eccentric loads.

9.3.1.2 *Design for flexure with or without axial loads*
Columns subjected to combined flexure and axial loads shall be designed in accordance with the principles of 10.2 including the effects of slenderness except as modified by the provisions of 9.3.1.

9.3.1.3 *Design for shear and torsion*
Columns subjected to shear and/or torsion shall be designed under the provisions of 10.3 except as modified by the provisions of 9.3.1.

9.3.1.4 *Beam-column joints*
Columns that form part of a beam-column joint shall be designed in accordance with section 11.

9.3.2 *Column construction*

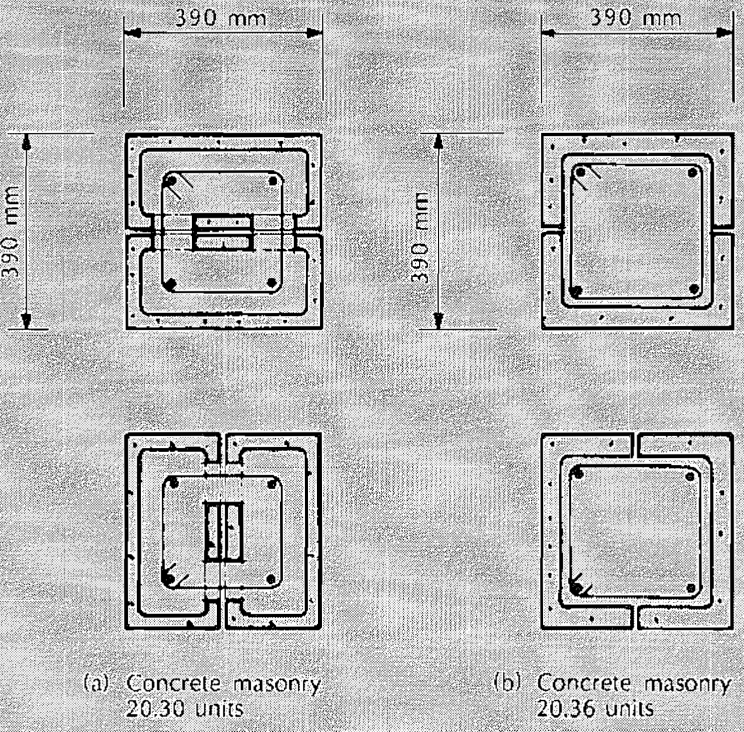
9.3.2.1 *Grouting columns*
All masonry columns shall be completely grouted.

9.3.2.2 *Minimum grout space*
The following requirements shall be observed for grout spaces:

(a) Columns shall be constructed so that the vertical grout spaces shall have a minimum clear dimension of 60 mm and a minimum area of 5 400 mm^2.

(b) A column grout space containing 4 bars shall have minimum clear grout space dimensions of 150 mm x 150 mm.

C9.3.2.2
Masonry columns may be constructed from various combinations of units (see figure 9.1). Where a column is made up with more than one vertical flue, the reinforcement limits of 9.3.5 shall apply to individual flues. Where 4 bars are to be contained within one flue the minimum dimensions are given in 9.3.2.2.



(a) Concrete masonry 20.30 units

(b) Concrete masonry 20.36 units

Figure 9.1 – Typical construction of masonry columns showing alternating courses

9.3.3 *Dimensional limitations of columns*

The minimum dimension of a column shall be not less than 240 mm nor the dimension given by either 0.05 L_n or 9.4.2.

C9.3.3

For cantilever columns, L_n shall be twice the height of the column above the base.

9.3.4 *Effective width for pilasters*

Pilasters may be designed as T-beams provided the net design axial load N^* does not exceed $0.1\phi f'_m A_m$. The limitations of 8.3.5 and 8.4 shall apply.

9.3.5 *Longitudinal reinforcement in columns*

9.3.5.1 *Maximum diameter of longitudinal reinforcement*

The diameter of longitudinal reinforcement used in a column shall not exceed:

(a) One quarter of the least dimension of the grout space or cavity containing reinforcement; nor

(b) One twelth of the least column dimension.

9.3.5.2 *Maximum area of reinforcement in grout space*

The maximum area of longitudinal reinforcement in a grout space or cavity shall not exceed $10/f_y$ times the area of the grout space or cavity, except that at laps the total area of reinforcement may be $15/f_y$ times the area of grout space or cavity.

9.3.5.3 *Minimum longitudinal reinforcement*
The minimum longitudinal reinforcement for a column shall be 4 bars in a rectangular arrangement whose total area exceeds the greater of:

(a) 400 mm^2; or

(b) $1.4\ A_m / f_y$.

9.3.6 *Transverse reinforcement in columns*
All columns shall have minimum transverse reinforcement to the following provisions.

9.3.6.1 *Minimum diameter of hoop reinforcement*
Rectangular hoop or tie reinforcement for columns shall be at least 6 mm in diameter for longitudinal bars less than 20 mm in diameter, and at least 10 mm in diameter for longitudinal bars being 20 and 25 mm in diameter and for bundled longitudinal bars.

9.3.6.2 *Hoops to enclose all longitudinal bars*
Rectangular hoop or tie reinforcement shall enclose all longitudinal bars.

9.3.6.3 *Spacing of hoop reinforcement*
Ties for columns shall be placed as follows:

(a) Centre-to-centre spacing of ties along the component shall not exceed the smaller of the least lateral dimension of the cross-section of a component, 0.5*h*, 16 longitudinal bar diameters, or 48 bar diameters;

(b) Ties shall be arranged so that every corner bar and at least every alternate longitudinal bar is laterally supported by a corner of a tie with an included angle of not more than 135°. No unsupported longitudinal bar shall be further than 150 mm clear on each side along the tie from a laterally supported bar." Amd 1 Dec '06

9.3.6.4 *Hoop required at changes in direction of longitudinal reinforcement*
Where longitudinal bars are offset, the slope of the inclined portion of the bar with the axis of the column shall not exceed 1 in 6, and the portions of the bar above and below the offset shall be parallel to the axis of the column. Adequate horizontal support at the offset bends shall be treated as a matter of design, and shall be provided by ties, spirals or parts of the floor construction. Ties or spirals so designed shall be placed not more than 150 mm from the point of bend. The horizontal thrust to be resisted shall be assumed as 1.5 times the horizontal component of the nominal force in the inclined portion of the bar, assumed to be stressed to f_y.

C9.3.6.4
Offset bending of bundled bars is prohibited for practical reasons.

9.3.6.5 *Rectangular hoop and tie reinforcement for large columns*
Spacing and arrangement of tie reinforcement in masonry units providing a grout space with a cross-section area of 50 000 mm^2 or more shall conform with NZS 3101.

C9.3.6
Where columns do not have a clear central flue, masonry unit webs and walls should be cut down to allow the grout to fully enclose the ties (see figure 9.1). The placing of ties in the column bed joints is not permitted.

9.3.7 *Column maximum design axial load*

For columns, the maximum design axial load in compression, N^* at a given eccentricity shall not exceed $0.7\phi N_o$ where:

$$N_o = 0.85\, f'_m\, (A_m - A_{st}) + f_y A_{st} \quad \text{(Eq. 9-1)}$$

9.3.8 *Column design loads*

Columns shall be designed for the most unfavourable combination of design moment, M^*, and design axial load, N^*.

9.3.9 *Approximate evaluation of slenderness effects for columns braced against sidesway*

The maximum design moment, M^*, shall be magnified for slenderness effects in accordance with the requirements of NZS 3101, except that for masonry the value of EI may be calculated from:

$$EI = \frac{E_m I_g}{(1+\beta_d)^4} \quad \text{(Eq. 9-2)}$$

9.4 Principles and requirements additional to 9.3 for structures designed using a limited ductile or ductile seismic design philosophy

9.4.1 *Design to be compatible with principles in section 3*

Amd 1 Dec '06

In the design of earthquake resisting ductile columns forming part of a limited ductile or ductile frame subjected to seismic actions at the ultimate limit state, the requirements of 3.7.3 for limited ductile columns and 3.7.4 for ductile columns shall be satisfied.

9.4.2 *Dimensional limitations for columns*

9.4.2.1 *Depth, width and clear span limitations for columns with moments at each end*

Depth, width, and clear length between the faces of supports of components with rectangular cross-section, to which moments are applied at both ends by adjacent beams, columns, or both, shall be such that:

$$L_n / b_w \leq 20 \quad \text{(Eq. 9-3)}$$

and

$$L_n h / b_w^2 \leq 80 \quad \text{(Eq. 9-4)}$$

9.4.2.2 *Depth, width and clear span limitations for cantilevered columns*

Depth, width, and clear length from the face of support of cantilever components with rectangular cross-section, shall be such that:

$$L_n / b_w \leq 10 \quad \text{(Eq. 9-5)}$$

and

$$L_n h / b_w^2 \leq 40 \quad \text{(Eq. 9-6)}$$

9.4.2.3 *Dimension of beam-column joints*

Dimensions of joints between beams and columns shall comply with the minimum dimensions set out in 11.4.2.

9.4.3 *Reinforcement in columns*

9.4.3.1 *Extent of potential plastic hinge region*

Potential plastic hinge regions in columns shall be considered to be the end regions adjacent to moment resisting connections over a length from the face of the connection as follows:

(a) Where $N^* \leq 0.3 \phi f'_m A_m$, not less than the larger of the section depth or the region of the component where the moment exceeds 0.8 of the maximum moment at that end of the component;

(b) Where $N^* > 0.3 \phi f'_m A_m$, not less than the larger of 1.5 times the section depth or the region of the component where the moment exceeds 0.7 of the maximum moment at that end of the component.

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9.4.3.2 *Longitudinal reinforcement in potential plastic hinge regions*

Throughout the column, the following longitudinal reinforcing requirements shall apply:

(a) Where $N^*_e < 0.1 \phi f'_m A_m$ the column shall be reinforced in accordance with 7.4.5.1 and 7.4.5.2;

(b) Where transverse reinforcement which complies with 9.3.6 is provided throughout the column, the maximum longitudinal reinforcement within the potential plastic hinge region shall be such that the depth from the extreme compression fibre to the neutral axis shall not exceed c_{max}, where:

$$c_{max} = \frac{h^2}{\mu L_n} \qquad \text{(Eq. 9-7)}$$

(c) Where transverse reinforcement is provided throughout the column which complies with NZS 3101 the maximum longitudinal reinforcement shall not exceed the limits given in 9.3.5.2.

C9.4.3.2

Principal reinforcement terminating within the potential plastic hinge regions of columns increases the moment capacity over that part of the hinge zone affected by the bar termination and reduces the length of the hinge region. This has the effect of reducing the hinge length and the overall ductility of the structure.

The requirement of 8.4.4.2 for anchorage of beam reinforcement within columns will mean that column proportions will be similar to short walls. At low axial loads, therefore, these column sections may be reinforced as for structural walls.

The construction of reinforced masonry columns whose length is greater than 600 mm is restrictive in the placing of column confinement ties. Until alternative means of confinement for columns have been tested, for example, the joint confining plates of 7.4.6.5, then the required available curvature ductility in the column must be attained by limiting the design depth of c or confining to NZS 3101.

9.4.3.3 *Rectangular hoop and tie reinforcement for columns*

In masonry columns, which provide a single grout space having a cross-sectional area of not less than 50 000 mm^2, the area and limits on spacing and arrangement of rectangular hoops with or without supplementary cross-ties in components shall conform to NZS 3101.

9.4.3.4 *Minimum diameter of supplementary cross-ties*

The diameter of supplementary cross-ties shall be not less than the diameter of the peripheral hoop.

9.4.3.5 *Details of hoop and tie reinforcement*

Hoop reinforcement shall be anchored by at least a 135° stirrup hook. Alternatively the ends of the hoop bar shall be welded to develop the breaking strength of the bar or $1.6f_y$, whichever is smaller. Each end of a supplementary cross-tie shall engage a longitudinal bar with at least a 135° stirrup hook.

9.4.4 *Column maximum design axial load*

For columns, the maximum design axial load in compression N^*_e, at a given eccentricity, shall not exceed $0.6\ \phi f'_m A_m$ unless it can be shown that N^*_e is less than $0.6\ \phi N_o$ where:

$$N_o = 0.85\ f'_m(A_m - A_{st}) + f_y A_{st} \qquad \text{(Eq.9-8)}$$

9.4.5 *Column load magnification factors*

For frames where sidesway mechanisms with potential plastic hinges forming only in columns are not permitted by AS/NZS 1170, the design moments and axial loads on columns shall include the effect of possible beam overstrength, and magnification of column moments due to dynamic effects, in order to provide a high degree of protection against hinging of the columns. Design shall follow the principles set out in NZS 3101.

10.2 Flexure with or without axial load

10.2.1 *Scope*

The provisions of this clause shall apply to the design of components for flexure with or without axial loads.

10.2.2 *General design principles and requirements for all structures*

10.2.2.1 *General principles*

Strength design of components for flexure with or without axial loads shall be based on assumptions given in 10.2.2.2 to 10.2.2.6 and on satisfaction of applicable conditions of equilibrium and compatibility of strains.

10.2.2.2 *Strain distribution*

Strain in reinforcement and in masonry shall be assumed directly proportional to the distance from the neutral axis, except that for deep beams with overall depth to clear span ratios greater than 2/5 for continuous spans and 4/5 for simple spans, a non-linear distribution of strain shall be considered (see 8.3.13).

10.2.2.3 *Maximum masonry strain*

The maximum usable strain at the extreme compression fibre shall be 0.003 for unconfined masonry and 0.008 for masonry confined in accordance with the requirements of 7.4.6.5.

C10.2.2.3

The value of the unconfined ultimate masonry compression strain has been set at 0.003. This is consistent with values adopted in other masonry design standards and with reported experimental data. In NZS 4230:1990 a value of 0.0025 was adopted on the basis that concrete masonry had a lower ultimate compression strain than plain concrete. With no data to support a value other than that for plain concrete, the value adopted corresponds to that employed in NZS 3101 for structural concrete design.

Limited testing$^{10.1}$ *has been carried out on confined concrete masonry and it is necessary therefore to limit the conditions under which the 0.008 strain can confidently be attained to the specific type of construction given by 7.4.6.5.*

10.2.2.4 *Reinforcing steel stress*

Stress in reinforcement below the lower characteristic yield strength, f_y, for the grade of reinforcement used shall be taken as E_s times steel strain. For strains greater than that corresponding to f_y, stress in reinforcement shall be considered independent of strain and equal to f_y.

10.2.2.5 *Tensile strength of masonry neglected*

Tensile strength of masonry shall be neglected in flexural calculations of reinforced masonry, except when meeting the requirements of table A1.

10.2.2.6 *Masonry stress/strain relationship*

The relationship between masonry compressive stress distribution and masonry strain may be assumed to be as follows:

(a) Masonry stress/strain condition – no confining plates

For an extreme fibre compression strain of 0.003 corresponding to unconfined masonry an equivalent rectangular masonry stress distribution may be used as defined by the following:

  (i) Masonry stress of 0.85 f'_m shall be assumed uniformly distributed over an equivalent compression zone bounded by the edges of the cross-section and a straight line located parallel to the neutral axis at a distance of $a = 0.85c$ from the fibre of maximum compressive strain;

(ii) Distance c from the fibre of maximum compressive strain to the neutral axis shall be measured in a direction perpendicular to that axis.

(b) Masonry stress/strain condition – confining plates
For an extreme fibre compressive strain of 0.008 for masonry confined in accordance with the requirements of 7.4.6.5 use of an equivalent rectangular masonry stress distribution may be used as defined by the following:

(i) Masonry stress of $0.9\,Kf'_{\mathrm{m}}$ shall be assumed uniformly distributed over an equivalent compression zone bounded by edges of the cross-section and a straight line located parallel to the neutral axis at a distance $a = 0.96c$ from the fibre of maximum compression strain where:

$$K = 1 + p_{\mathrm{s}}\frac{f_{\mathrm{yh}}}{f'_{\mathrm{m}}} \quad \text{(Eq. 10-1)}$$

(ii) Distance c from the fibre of maximum compression strain to the neutral axis shall be measured in a direction perpendicular to that axis.

(c) Alternative stress/strain relationships
Alternatively, the relationship between masonry compressive stress distribution and masonry strain may be assumed to be rectangular, trapezoidal, parabolic or any other shape that results in prediction of strength in substantial agreement with results of comprehensive tests.

C10.2.2.6
The stress distribution for unconfined masonry is the same as that given in NZS 3101, and has been found to give reasonable agreement between the predicted strength of sections and the results of tests. Tests on confined masonry reported in Reference 10.1 showed that at compressive strains approaching 0.008 the stress distribution of 10.2.2.6(b) more accurately predicted the nominal flexural strength.

10.2.2.7 *Balanced strain conditions*
Balanced strain conditions exist at a cross-section of a beam when the centroid of tensile force provided by reinforcement reaches the strain corresponding to its lower characteristic yield strength, f_{y}, just as masonry in compression reaches its assumed ultimate strain of 0.003.

10.2.2.8 *Bearing strength*
The nominal bearing strength of masonry shall be taken as not greater than $0.65\,f'_{\mathrm{m}}A_1$.

C10.2.2.8
Bearing surfaces in masonry may be partly block or grout. The factor for the allowable bearing stress in masonry is reduced from the value used in NZS 3101 to allow for various combinations of bearing materials. Though not excluded by this clause it is recommended that elements bearing on masonry should not rely solely on the bearing of concrete face-shells.

10.2.3 *Principles and requirements additional to 10.2.2 for structural component design using a limited ductile or ductile design philosophy*

10.2.3.1 *Flexural strength for limited ductile masonry*
Where capacity design principles are not applied for limited ductile structures the flexural strength provided outside the designated potential plastic hinge regions shall be such that:

$$\phi M_{\mathrm{n}} \geq M^{*}_{\mathrm{G}} + M^{*}_{\mathrm{Qu}} + 1.5\,M^{*}_{\mathrm{E}} \quad \text{(Eq.10-2)}$$

Alternatively, capacity design principles may be applied as specified in 3.7.3.2.

10.3.2.7 *Shear stress provided by masonry under axial load*

The shear strength enhancement resulting from axial load is considered as an independent component of shear strength, resulting from a diagonal strut.

$$v_p = 0.9 \frac{N^*}{b_w d} \tan\alpha \qquad \text{(Eq. 10-7)}$$

where

(a) N^* shall not be taken greater than 0.1 $f'_m A_g$; and

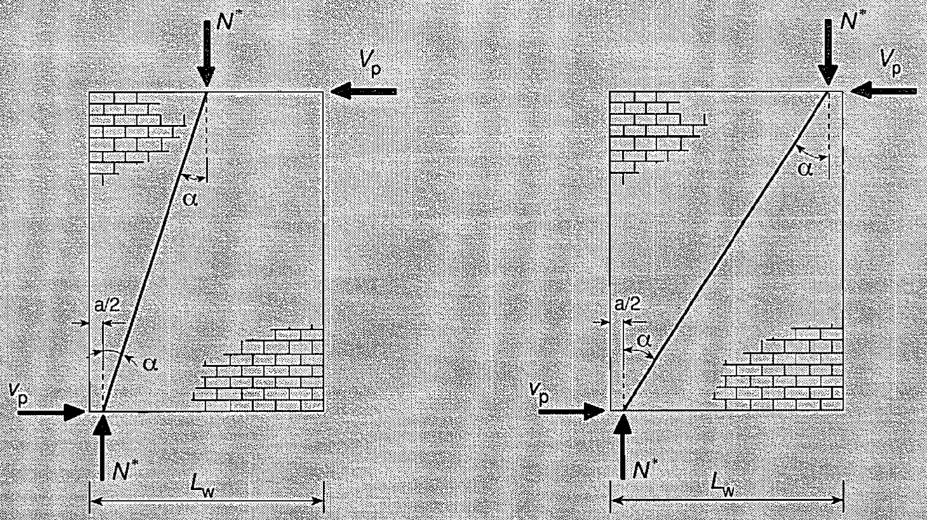
(b) v_p shall not be taken greater than 0.1 f'_m; and

(c) In determining the shear stress provided by axial load, v_p, effects of axial tension, including tension from creep and shrinkage and from differential temperature, shall be considered.

C10.3.2.7

It has been observed through experimental studies [10.4, 10.5, 10.6] that axial compression load has a significant influence on the in-plane shear strength of masonry components, mainly because it suppresses the tensile field in a material inherently weak in tension. As the axial compression load increases, so does the ability of the masonry components to offer shear resistance. However, it was also observed through experimental studies that the post-cracking deformation capacity of a masonry wall is significantly reduced with increasing axial compression load. This is because the failure type becomes more brittle as the axial compression stress increases. Consequently, the limitation of $N^* \le 0.1 f'_m A_g$ *is included in Equation 10-7 to prevent possible brittle shear failure of masonry components due to excess reliance upon* v_p *in Equation 10-5.*

The shear strength enhancement resulting from axial compression is considered as an independent component of shear strength, resulting from a diagonal strut[10.7]. Figure 10.2 illustrates the evaluation of v_p *for concrete masonry shear walls. For a cantilever wall,* α *is the angle formed between the wall axis and the strut from the point of load application to the centre of the flexural compression zone. For a wall in double bending,* α *is the angle between the wall axis and the line joining the centres of flexural compression at the top and bottom of the wall. This implies that the shear strength of squat axially loaded walls should be greater than that of more slender walls. Limitation of* $v_p \le 0.1 f'_m$ *is included in 10.3.2.7 to prevent excess dependence on* v_p *in a relatively squat masonry component.*



(a) Single bending **(b) Double bending**

Figure 10.2 – Contribution of axial load to wall shear strength

10.3.2.8 *Shear reinforcement*
Shear reinforcement may consist of:

(a) Stirrups or ties perpendicular to the axis of the component;

(b) Stirrups making an angle of 45° or more with the longitudinal tension bars;

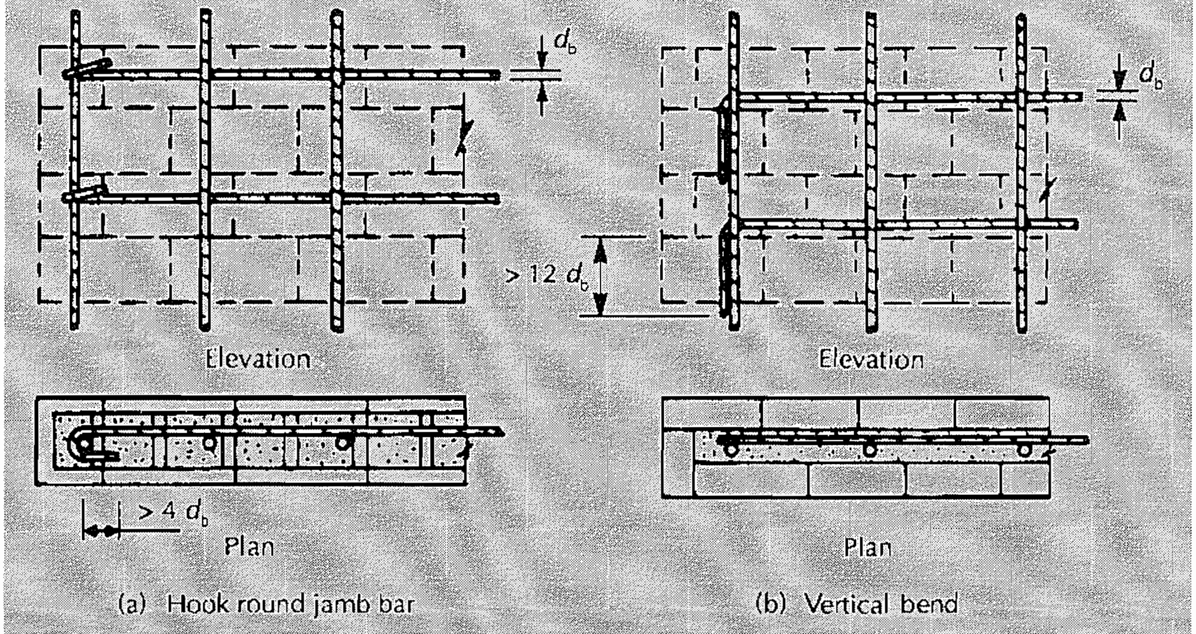
(c) Spirals;

(d) Horizontal wall reinforcement terminating in a standard hook or bend at each end.

C10.3.2.8
Many of the types of shear reinforcement listed in 10.3.2.8 are impractical with current methods of construction in masonry, and are only included so that future developments are not prejudiced by restrictive clauses.

10.3.2.9 *Shear reinforcement details*
Shear reinforcement shall extend to at least a distance, *d*, from the extreme compression fibre of the component and shall be anchored at both ends to develop the lower characteristic yield strength of reinforcement.

C10.3.2.9
It is essential that shear reinforcement be adequately anchored at both ends, to be fully effective on either side of any potentially inclined crack. This generally requires a hook or bend at the end of the reinforcement as shown in figure 10.3. Although hooking the bar round the end vertical reinforcement in walls is the best solution for anchorage, (as shown in figure 10.3(a)) it may induce excessive congestion at end flues and result in incomplete grouting of the flue. Consequently bending the reinforcement up or down into the flue as shown in figure 10.3(b) may be preferable, particularly for walls of small width.



(a) Hook round jamb bar (b) Vertical bend

Figure 10.3 – Anchorage of shear reinforcement in walls

10.3.2.10 *Spacing of shear reinforcement*

Spacing of shear reinforcement, placed perpendicular to the axis of the component, shall not exceed:

(a) $0.5d$ nor 600 mm for beams and columns;

(b) $0.5\ L_w$ for walls.

10.3.2.11 *Design of shear reinforcement*

The following shall be considered when designing shear reinforcement:

(a) When the required shear stress, v_n, exceeds the shear stress provided by masonry, v_m, and provided by axial load, v_p, then shear reinforcement shall be provided for the difference, $(v_n - v_m - v_p)$;

(b) When shear reinforcement perpendicular to the axis of the component is used, the required shear stress provided by shear reinforcement, A_v, within a distance, s, shall be not less than:

$$v_s = C_3 \frac{A_v f_y}{b_w s} \qquad \text{(Eq. 10-8)}$$

where

(i) For walls: $C_3 = 0.8$;

(ii) For beams and columns: $C_3 = 1.0$.

(c) Where shear reinforcement is required by 10.3.2.11(a) a minimum area shall be given by:

$$A_v = \frac{0.15 b_w s}{f_y} \qquad \text{(Eq. 10-9)}$$

C10.3.2.11

Equation 10-8 is based on the ACI method, as incorporated in NZS 3101. It was observed in an experimental study[10.5] that shear reinforcement has limited efficiency in masonry walls. It is assumed that the reduced efficiency of shear reinforcement is due to bar anchorage effects, and this condition is represented by the C_3 term incorporated in Eq. 10-8 which differentiates between walls which rely on bar development length and beams and columns, which generally have shear reinforcement comprised of closed hoops. Figure 10.1 (a) and (b) illustrate effective areas for in-plane shear for fully grouted and partially grouted walls.

10.3.2.12 *Components loaded in torsion*

For components loaded in torsion the following applies:

(a) If the torsion in the component is required to maintain the equilibrium in the structure, and if the magnitude of the required nominal strength exceeds $0.20\ A_{mo} t_m v_g$, where v_g is given in 10.3.2.4, torsional reinforcement shall be provided to the requirements of NZS 3101.

(b) If the torsion in the component arises because the component must twist to maintain compatibility, the effect of torsion on the component may be neglected, provided that the shears in the structure are computed assuming no torsional stiffness of the component. Minimum torsional requirements of NZS 3101 shall apply when the torsional shear stress exceeds $0.2\ v_g$, where v_g is given in 10.3.2.4.

C10.3.2.12

Masonry elements, being typically slender, are not particularly effective in resisting torsion. Consequently, whenever possible, torsion should not be relied upon as a mechanism for load transfer, to maintain equilibrium of part or all of the structure. Where torsional shear stresses result from compatibility of deformation requirements, it is conservative to ignore the contribution of torsional stiffness.

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In all cases the provisions of NZS 3101 apply. If torsional reinforcement is required in a masonry element, the element must be of sufficient width to allow two layers of reinforcement and fully enclosing rectangular stirrups to be placed. This would not be possible with most common forms of masonry construction, which emphasizes the need to design in such a way that torsional stresses remain below the maximum level of 0.2 v_g which can be carried by the masonry alone.

10.3.2.13 *Shear friction*

Provisions of this clause may be applied where it is appropriate to consider shear transfer across a given plane such as an existing or potential crack, or an interface between dissimilar materials.

(a) A crack shall be assumed to occur along the shear plane, with relative displacement along the assumed crack resisted by friction maintained by shear-friction reinforcement across the assumed crack. Shear-friction reinforcement shall be placed approximately perpendicular to the assumed crack.

(b) The required area of shear friction reinforcement A_{vf} shall be given by:

$$A_{vf} = \frac{1}{f_y}\left(\frac{V^*}{\mu_f \phi} - N^*\right) \quad \text{(Eq. 10-10)}$$

where the coefficient of friction, μ_f, shall be:

(i) For grout placed monolithically, or when placed against previously hardened concrete where the interface is clean, free of laitance, and intentionally roughened to a full amplitude of not less than 3 mm .... $\mu_f = 1.0$

(ii) For grout placed against hardened grout or concrete where the interface is clean and free of laitance but has not been intentionally roughened; or for grout placed against as-rolled structural steel in accordance with 10.3.2.15 .... $\mu_f = 0.7$

(c) Shear stress, v_n, shall not exceed the value v_g given in 10.3.2.4 and shall be calculated from the net area over a length of the component which does not exceed 300 mm each side of the centreline of reinforcement, nor the centre to centre spacing of the reinforcing crossing the potential crack.

C10.3.2.13

Virtually all considerations of shear are intended to prevent diagonal tension failures. There are instances in all structures where shear transfer is required by shear friction along a known or likely crack path. Unless the level of shear stress resulting from diagonal tension is high, or there are construction irregularities in normal wall construction resulting in reduced area at potential planes of shear sliding, it is unnecessary to check the shear friction stresses at horizontal joints in walls.

Shear friction in masonry, where the interlocking materials have relatively small grain size, is not effective in regions where wide cracks may develop. Examples of areas where shear friction should be checked are shown in figure 10.4.

Note that in figure 10.4(c) shear will need to be transmitted across the control joint only if the wall is designed to act as a single entity under lateral in-plane loads. If two portions of wall separated by the control joint are allowed relative movements, and lateral in-plane loads are carried independently by the two sections in proportion to their individual stiffnesses, shear friction provisions across the control joint do not apply.

Resistance to shear sliding along a potential shear failure plane is provided by frictional forces between the sliding surfaces. The frictional forces are proportioned to the coefficient of friction, μ_f*, and the total normal force acting across the joint, which may be provided by axial force,* N^**, and distributed reinforcement,* $A_{vf}\,f_y$*. Note that for horizontal planes in walls,* A_{vf} *may be provided by vertical flexural reinforcement.*

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The effective clamping force across the crack will be $A_v\,f_y + N^*$*. Thus the design shear force,* ϕV_n *that can be transmitted across the crack by shear friction is* $\mu_f\,(A_v\,f_y + N^*)$*, which leads to Equation 10-10 for the required area of reinforcement. Note that if a tension force is present,* N^* *will be negative, increasing the requirement for reinforcement. In using Equation 10-10 care must be taken to ensure that* N^* *is the axial load arising from the same combination of load cases as used in calculating the design shear force,* V^**.*

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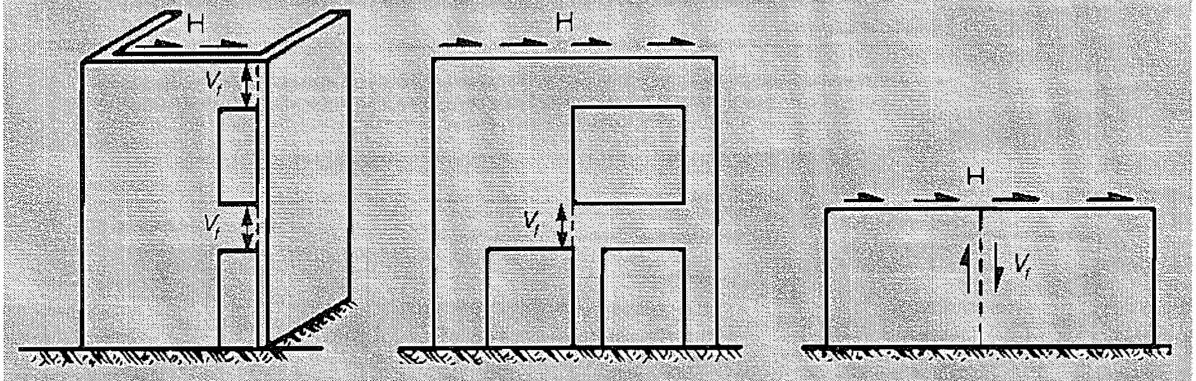
Maximum permitted shear friction stress is limited to the grade-dependent value allowed for shear resulting from diagonal tension. In computing the shear friction stress, the net masonry area up to 300 mm away from a reinforcing bar may be considered to be effective. Portions of the interface further than this from a bar are unlikely to benefit from the clamping action of reinforcement.

The coefficients of friction listed in 10.3.2.13 are generally lower than those applicable to concrete because of the reduced roughness of potential crack surfaces compared to concrete crack surfaces.



(a) Junction of walls (b) Face of support beams (c) Control joint in long wall

Figure 10.4 – Examples of locations where shear friction should be checked

10.3.2.14 *Detailing of shear-friction reinforcement*
Shear-friction reinforcement shall be evenly distributed across the assumed crack and shall be adequately anchored on both sides by embedment, or, where adequacy is established by special study, by welding to special devices.

10.3.2.15 *Shear transfer between masonry and steel*
When shear is transferred between masonry and rolled steel, steel shall be clean and free of paint or other material detrimental to the maintenance of a sound bond.

10.3.2.16 *Special provisions for deep beams*
The provisions of NZS 3101 shall apply, except that v_c shall be replaced by v_m throughout, where v_m is given in 10.3.2.5 and 10.3.2.6.

10.3.2.17 *Special provisions for brackets and corbels*
For brackets and corbels the provisions of NZS 3101 shall apply.

10.3.2.18 *Special provisions for shear from face loads*
The following special provisions apply to shear from face loads:

(a) Total shear stress, v_n, shall be calculated from the area equal to the effective depth, d, multiplied by the width of the filled grout spaces, b_w, (see figure 10.1) as follows:

(i) For fully grouted construction in running bond, b_w shall be the lesser of s_L, or $4t$;

(ii) For fully grouted construction in stack bond, b_w shall be the lesser of s_L, or $3t$;

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(iii) For partially grouted construction, b_w shall be equal to the width of the grouted cells containing longitudinal reinforcement resisting face load bending moment.

(b) Total shear stress, v_n, calculated from the effective area defined in (a) shall not exceed v_m, as follows:

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(i) Observation Type A masonry: v_m = 0.30 MPa or $0.3(0.1\ f'_m + N^*/A_g)$ whichever is the greater, except that $0.1\ f'_m$ shall not be taken greater than 1.6 MPa and that $(0.1\ f'_m + N^*/A_g)$ shall not be taken to exceed 2.4 MPa;

(ii) Observation Type B masonry: $v_m = 0.24 + 0.3\ N^*/A_g$ but $\leq$ 0.56 MPa;

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(iii) Observation Type C masonry: v_m = 0.15 MPa;

Unless:

(A) Shear reinforcement is provided to carry the excess shear, $(v_n - v_m)$; or

(B) It can be demonstrated with reference to the provisions of 10.3.2.13 that, for the case of a face-loaded cantilever wall which is not required to exhibit ductility in the in-plane direction, sufficient shear friction can be mobilized to resist v_n.

(c) Shear strength required to resist face loads in the vicinity of concentrated loads or reactions shall be in accordance with the special provisions for slabs and footings in NZS 3101.

C10.3.2.18
The effective area in resisting shear, defined in this clause, is described further in figure 10.1. The diagrams (c) and (d) show provisions for face loading in a vertical plane. If face loading is being considered to act horizontally, stack bond provisions should be applied.

The provisions for face load shear are based on the values previously specified in NZS 4230:1990. No comprehensive research or testing of face load shear was undertaken for that Standard, nor since.

Some minor modifications have been made to the previous provisions, now neglecting the shear contribution of adjoining webs of masonry units in partial fill masonry in recognition of the widely used current practice of not completely mortar filling between webs of adjoining masonry units. Additionally, the definition of longitudinal reinforcement spacing, s_L, has been clarified to recognize that face loadings may exist in both horizontal and vertical planes.

If face load shear stress exceeds that which can be provided by masonry shear resisting mechanisms, special shear reinforcement must be provided. In general this will be impractical and redesign will be necessary to reduce shear stresses to acceptable levels. In the particular case of a cantilevered wall, as might be used for retaining purposes, the shear-friction generated by way of the compressive stress resultant which equilibrates the tensile stresses in the flexural reinforcement may be invoked as a viable shear-resisting mechanism.

10.3.3 *Principles and requirements additional to 10.3 for components designed using a limited ductile or ductile seismic design philosophy*

10.3.3.1 *Shear strength for limited ductile masonry*

Where capacity design principles are not applied for limited ductile structures, shear strength shall have a suitable margin over the required flexural strengths, such that:

$$\phi V_n \geq V_G^* + V_{Qu}^* + 2V_E^* \qquad \text{(Eq. 10-11)}$$

Alternatively, capacity design principles may be applied as detailed in 3.7.3.2.

10.3.3.2 *Shear strength*

In designing for shear strength the following shall be considered:

Amd 1 Dec '06 (a) The design shear force in components subjected primarily to flexure shall be determined from considerations of transverse actions on the component, with the flexural overstrength being developed at the most probable location of critical sections within the component or in adjacent components, together with the gravity load at the appropriate load factor.

Amd 1 Dec '06 (b) The design shear force in components subjected to combined flexure and axial load shall be determined from considerations of static actions on the component, with the worst likely combination of the maximum likely end moments, and where appropriate with flexural overstrength being developed at critical sections.

(c) In applying Equation 10-3 to the design of cross-sections where the shear force is calculated in accordance with (a) and (b), a strength reduction factor $\phi = 1.0$ shall be adopted.

C10.3.3.2

The shear strength of masonry elements degrades with inelastic flexural cycling, and it is thus necessary to adopt more conservative provisions for shear design when significant flexural ductility is required at the design level of seismic attack.

In order to guard against premature shear failure of components, the shear strength required for limited ductile and ductile structures is assessed on the basis of equilibrium conditions existing on the attainment of flexural overstrengths. Refer to the appropriate sections of NZS 3101 for further guidance.

Tests have shown that walls that fail in shear have poor cyclic response and that their strength deteriorates rapidly. Hence in the potential plastic hinge region of a wall it is usually required that the shear reinforcement be designed to carry the entire shear load. A less conservative approach adopted here has assumed that little strength degradation occurs up to a component ductility ratio of 1.25, followed by a gradual decrease to higher ductility. This behaviour is represented by figure 10.5 and recognized by the v_{bm} employed in table 10.1.

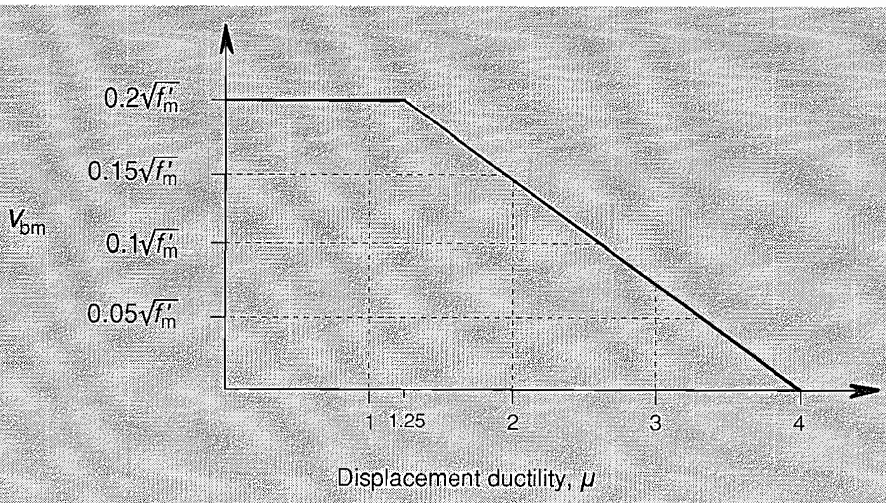

Figure 10.5 – Relationship between ductility and masonry shear resisting mechanism

10.3.3.3 *Shear strength provided by masonry*
In all potential plastic hinge regions of beams, columns, and walls the masonry shear stress v_m shall be as detailed in 10.3.2.5 and 10.3.2.6.

10.3.3.4 *Length of potential plastic hinge region*
The potential plastic hinge length is given in 7.4.3 for walls, in 8.4.3.1 for beams, and in 9.4.3.1 for columns. For detailing purposes the length of the potential plastic hinge regions of a component, shall be taken as the extent of the potential plastic hinge for flexure.

10.3.3.5 *Shear reinforcement details*
Within potential plastic hinge regions, the maximum spacing of shear reinforcement perpendicular to the axis of the component shall not exceed $h/4$ for beams and columns, and $L_w/4$ for walls.

C10.3.3.5
The provisions of 10.3.3.5 apply only within the potential plastic hinge region defined by 7.4.3 for walls, 8.4.3.1 for beams and 9.4.3.1 for columns. Outside such areas, the conventional provisions of 10.3.2.10 may be used. Within beams and columns, shear reinforcement would normally be spaced not further apart than one quarter of the depth, and must be perpendicular to the axis of the component.

10.3.3.6 *Openings in the web*
The placement of openings in the web of flexural components shall be such that potential failure planes across such openings cannot occur.

(a) Small openings
Small openings shall be defined and restricted by the following:

(i) Small rectangular or circular openings may be placed in the mid-depth of the web provided that cover requirements to longitudinal and transverse reinforcement are satisfied, and the clear distance between such openings, measured along the component, is not less than 400 mm. The maximum dimensions of a small opening in any direction within the plane of a component shall not exceed one eighth of the total depth or one eighth of the unsupported length of the component.

(ii) Webs with openings larger than that permitted by (i) shall be subject to special study to ensure that the forces and moments are adequately transferred in the vicinity of the openings.

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(b) Large openings

Large openings shall be defined and restricted by the following:

(i) An opening is considered large where the largest dimension of an opening exceeds one quarter of the total depth or one quarter of the unsupported length of the component. Except as provided by (iv) the height of openings in beams and the width of openings in columns or walls shall not exceed 0.33 h nor shall the edge of an opening be closer than 0.3 h to the compression face of the component. Such openings shall not be placed in the web where they could affect the flexural or shear capacity of the component, nor where the total shear exceeds 0.5 v_g, nor in potential plastic hinge regions unless designed to (iv).

(ii) For openings defined by (i), longitudinal and transverse reinforcement shall be placed in the compression side of the web at one side of the opening to resist 1.5 times the shear and moment generated by the shear across the opening. Shear transfer in the tension side of the web on the other side of the opening shall be neglected.

(iii) Transverse web reinforcement, extending over the full depth of the web, shall be placed adjacent to both sides of a large opening over a distance not exceeding half of the effective depth of the component to resist twice the entire design shear across the opening.

(iv) Where large openings occur within potential plastic hinge regions, or where openings exceed the dimensional limitations of 10.3.3.6 (a)(i), the modification to structural behaviour of the component shall be determined by a special study, and the component shall be designed to transmit the seismic actions in the modified form.

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C10.3.3.6

This clause gives guidance on how to assess the significance of openings in webs of masonry components. Provided such openings satisfy the dimensional limitations of 10.3.3.6(a)(i), the influence on structural behaviour will be negligible. Larger openings should be designed by rational analysis, in accordance with 10.3.3.6(b)(iv), or the generally more stringent requirements of 10.3.3.6(b)(i) to 10.3.3.6(b)(iii) should be satisfied.

Large openings result in changes to the structural form. It is particularly important that these effects be properly assessed where they occur within a hinging region, as local hinging beside, above, or below the opening may dominate behaviour. Consequently a rational analysis taking into account the stiffness of pier and spandrel units resulting from the opening, must be carried out to establish the expected structural behaviour.

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11 BEAM-COLUMN JOINTS

11.1 Notation

A_{jh}	Total area of effective horizontal joint shear reinforcement, mm^2
A_{jv}	Total area of effective vertical joint shear reinforcement, mm^2
b_c	Overall width of column, mm
d_{bb}	Diameter of longitudinal beam reinforcement passing through joint, mm
d_{bc}	Diameter of longitudinal column reinforcement passing through joint, mm
f_y	Reinforcement yield strength, MPa
h_b	Overall depth of beam, mm
h_c	Overall depth of column, mm
v_g	Maximum permitted type-dependent total shear stress, defined in 10.3.2.4, MPa
v_{jh}	Nominal horizontal shear stress in joint core, MPa
v_m	Maximum permitted grade-dependent shear stress provided by masonry, defined in 10.3.3.3, MPa
V_{jh}	Total horizontal shear force across a joint, N
V_{jv}	Total vertical shear force across a joint, N
V_{mh}	Nominal horizontal joint shear strength provided by masonry shear resisting mechanism only, N
V_{mv}	Nominal vertical joint shear strength provided by masonry shear resisting mechanism only, N
V_{sh}	Nominal horizontal joint shear strength provided by horizontal joint shear reinforcement, N
V_{sv}	Nominal vertical joint shear strength provided by vertical joint shear reinforcement, N
ϕ	Strength reduction factor as specified in 3.4.7

11.2 Scope

Provisions of this section apply to the design of masonry beam-column joints, including the joint zone of intersecting masonry wall members, which are subject to shear induced by gravity or earthquake loads or both. Only one-way unconfined masonry beam-column joints are considered in this section.

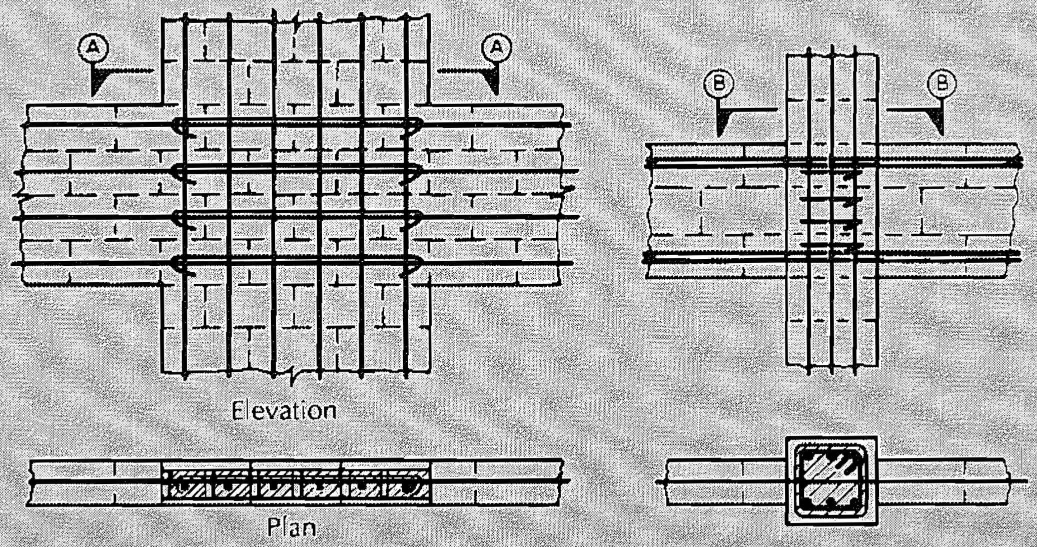
C11.2

This section covers the design of beam-column joints, and follows the format of the corresponding section of NZS 3101. Provisions for joints are limited to one-way unconfined joints only. An example is shown in figure 11.1(a). It should be noted that one-way masonry joints will generally be planar, as shown in figure 11.1(a), with the vertical support component being a wall rather than a masonry column. This results from dimensional limitations of 11.4.2 which follows in the depth, h_c, of the vertical support component exceeding those suitable for column proportions, particularly where the joint design is governed by seismic loading. Two-way joints should be designed with the joint constructed of reinforced concrete. In this case, design of the joint will be covered by provisions of NZS 3101. In this section the term 'joint' is in all cases taken to mean the beam-column junction. Thus 'joint reinforcement' refers to reinforcement placed in the beam-column junction.

It is possible to design a confined masonry beam-column joint using pilaster units to form the column and joint. A common example, using 400 x 200 concrete masonry pilaster units, is illustrated in figure 11.1(b). With such a design the core of the joint is effectively reinforced grout, or concrete, and the joint may be designed to the provisions of section 11 of NZS 3101. Note that this will require a minimum of four column longitudinal bars, confining hoops in the joint, and horizontal and vertical joint shear reinforcement. Where the design is such that under seismic loading, potential plastic hinges may form in the beams at the column faces, requirements for anchorage of reinforcement of NZS 3101 would limit the beam bar diameter size for the joint in figure 11.1(b) to 16 mm for reinforcement having $f_y = 300$ *MPa and 10 mm for reinforcement having* $f_y = 500$ *MPa. With such a design the column should be filled with concrete rather than grout to ensure satisfactory conditions for bond transfer from beam reinforcement to the joint.*

It needs to be recognized that the behaviour of one-way joints in masonry can be expected to be quite different from that of one-way concrete joints. The prime difference in behaviour lies in the fact that beam and column reinforcement on a one-way joint will generally lie in a single plane, and lateral confinement of the joint masonry will be difficult, if not impossible, to achieve. Consequently the very high bond stresses necessary to transmit rapid variation in beam reinforcement stress across the joint in concrete joints under lateral loading cannot be relied upon in masonry. The approach taken has been to assume that conditions for bond transfer across the joint, and for shear transfer within the joint, are similar to conditions applying for bond and shear in masonry walls.

The commentary presented in this section is intended to be additional to that provided in NZS 3101, to which reference should be made for detailed evaluation of joint forces, and description of joint behaviour.



(a) Planar beam-wall joint (unconfined) using open-end bond beam units (Joint core designed to provisions of this Standard)

(b) One-way beam-column joint using pilaster units for column joint (confined) (Joint core designed to provisions of NZS 3101)

Figure 11.1 – Masonry interior beam column joints

11.3 General principles and requirements for all structures

11.3.1 *General design principles*
Beam-column joints shall satisfy the following criteria:

(a) A joint shall perform under service loads at least as well as the components that it joins;

Amd 1 Dec '06 (b) A joint shall have a design strength sufficient to resist the most adverse load combinations sustained by the adjoining components, as specified by AS/NZS 1170.

C11.3.1
The basic requirements of a beam-column joint are:

(a) It must perform satisfactorily under service loads;

(b) Its strength should not normally govern the strength of the structure; and

(c) Its behaviour should not impede the development of the full strength of the adjoining components.

Other important requirements are:

(i) Ease of construction; and

(ii) Access for depositing and compacting concrete.

11.3.2 *Dimensional limitations*

11.3.2.1 *General requirements*
Dimensional limitations for the joint shall conform to dimensional limitations applicable to all components that it joins.

C11.3.2.1
Width and height of the joint should be checked to ensure bond conditions on beam and column longitudinal reinforcement do not exceed those implied by the anchorage requirements of section 6.

11.3.2.2 *Limitations for seismic design*
Joints for which elastic or nominally ductile seismic load combinations govern design shall also be subject to the dimensional limitations of 11.4.2.

C11.3.2.2
The structural demand on joints is greatly dependent on the type of loading, and therefore design procedures are necessarily appropriate to the severity of each type of loading. Where static gravity loading governs, strength under monotonic loading without stress reversals will be the design criterion. Seismic loading is more severe because strength degradation of the concrete in the joint may occur under repeated reversed loading, and a large amount of joint reinforcement is therefore required. Bond conditions for beam reinforcement passing through joints dominated by seismic loading will be more severe than gravity-load dominated joints due to the reversal of stress from tension to compression across the joint.

The necessary limitations imposed by 11.4.2 on joint dimensions for bar anchorage should be considered at the preliminary design stage for masonry structures which are designed to 3.7.

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11.3.3 *Design actions*

The design actions on a beam-column joint shall be evaluated from the maximum stresses generated by all components meeting at the joint, subjected to the most adverse combinations of actions as required by AS/NZS 1170, with the joint in equilibrium.

C11.3.3

The joint must be designed to resist the forces considered in designing the components and in those combinations producing the most severe force distribution at the joint. Indirect actions resulting from time-dependent effects such as creep, shrinkage, or settlement should be considered.

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11.3.4 *Strength reduction factor, ϕ*

In determining the shear strength of the joint the value of the strength reduction factor, ϕ, shall be 0.75.

11.3.5 *Maximum permissible horizontal shear stress*

The nominal horizontal shear stress in the joint, v_{jh}, shall not exceed the grade-dependent value, v_g, specified in table 10.1 where:

$$v_{jh} = \frac{V_{jh}}{b_c h_c} \qquad \text{(Eq. 11-1)}$$

11.3.6 *Design principles*

The joint shear shall be assumed to be resisted by a masonry mechanism plus a truss mechanism, comprising horizontal and vertical stirrups or bars and diagonal masonry struts, in accordance with 11.3.7.1 and 11.3.7.2, except that corner joints of portal frame structures and in other applications, joints may be detailed by rational analysis so that shear forces are transferred by an acceptable mechanism and so that anchorage of the flexural reinforcement within the joint is assured.

C11.3.6

Joints may be designed using the relevant principles of force equilibrium. A rational analysis may be used to show the extent to which a principal diagonal compression strut can carry a proportion of the joint shear, the remainder being carried by horizontal and vertical or diagonal joint shear reinforcement. Alternatively Equations 11-2 to 11-9 may be used to evaluate this contribution.

11.3.7 *Horizontal joint shear reinforcement*

11.3.7.1 *Force to be resisted by horizontal shear reinforcement*

The horizontal design shear force to be resisted by the horizontal joint shear reinforcement shall be:

$$V_{sh} = \frac{V_{jh}}{\phi} - V_{mh} \qquad \text{(Eq. 11-2)}$$

where $V_{mh} = 0.5\, V_{jh}$ .... (Eq. 11-3)

but need not be taken as less than:

$$V_{mh} = v_m b_c h_c \qquad \text{(Eq. 11-4)}$$

where v_m is the grade-dependent masonry shear stress specified in 10.3.2.5.

➤

C11.3.7.1
These provisions make due allowance for the considerable contribution of the diagonal compressive strut in the masonry to joint shear transfer.

No allowance is made for increasing shear carried by masonry under increasing axial load.

11.3.7.2 *Area of horizontal shear reinforcement*
The horizontal shear reinforcement shall be capable of carrying the design joint shear force given by Equation 11-2 across the corner to corner potential failure plane. The effective total area of the horizontal reinforcement that crosses the critical diagonal plane shall be not less than:

$$A_{jh} = \frac{V_{sh}}{f_y} \quad \text{(Eq. 11-5)}$$

Horizontal stirrups shall be placed between the outermost layers of the top and bottom beam reinforcement and shall be distributed as uniformly as practicable. Any stirrup leg between bends around column bars that does not cross the potential failure plane shall be neglected. Stirrup legs shall anchor round the extreme column vertical bar at each end of the joint.

C11.3.7.2
The provisions of 11.3.7.2 for horizontal shear reinforcement are based on providing reinforcement to resist that portion of the joint shear not resisted by the diagonal masonry strut oriented corner-to-corner across the joint.

11.3.8 *Vertical joint shear reinforcement*

11.3.8.1 *Force to be resisted by vertical shear reinforcement*
The vertical design shear force to be resisted by the vertical joint shear reinforcement shall be:

$$V_{sv} = \frac{V_{jv}}{\phi} - V_{mv} \quad \text{(Eq. 11-6)}$$

where $V_{mv} = 0.6\ V_{jv}$ .... (Eq. 11-7)

but need not be taken as less than:

$$V_{mv} = v_m b_c h_c \quad \text{(Eq. 11-8)}$$

where v_m is the grade-dependent masonry shear stress specified in 10.3.2.5.

C11.3.8.1
To sustain a diagonal compression field by a truss mechanism, vertical joint shear reinforcement is also required. This can be computed in the same way as the horizontal joint shear reinforcement. The allowance for vertical shear carried on the masonry is greater than that for horizontal shear because of the beneficial effects of column axial load.

11.3.8.2 *Area of vertical shear reinforcement*
The requirements for the area of shear reinforcement are:

(a) The total area of vertical joint shear reinforcement within the effective joint width, b_c, shall be not less than:

$$A_{jv} = \frac{V_{sv}}{f_y} \qquad \text{(Eq. 11-9)}$$

(b) The vertical joint shear reinforcement shall consist of intermediate column bars, placed in the plane of bending between extreme bars, or vertical stirrup ties or special vertical bars, placed in the column and adequately anchored to transmit the required tensile forces within the joint.

11.4 Principles and requirements additional to 11.3 for joints in structures designed using a limited ductile or ductile seismic design philosophy

11.4.1 *General*
Special provisions are made in 11.4 for beam-column joints that are subjected to forces arising as a result of inelastic lateral displacements of limited ductile and ductile frames. Joints must be designed in such a way that the required energy dissipation occurs in potential plastic hinges of adjacent components and not in the joint core region. Joints in structures designed to lateral load levels corresponding to elastic or nominally ductile response in accordance with AS/NZS 1170 may be designed to the provisions of 11.3.

C11.4.1
Provisions are made for beam-column joints that are subjected to forces consistent with lateral loading on frames causing inelastic displacements. Particularly severe conditions can arise with respect to shear strength and anchorage of the reinforcement passing through or terminating in a joint. The basic requirement of the design is that joints must be somewhat stronger than adjacent hinging components, which are normally the beams. Because shear strength and the anchorage of the reinforcement controls joint design, energy dissipation within the joint core is undesirable. It can lead to rapid loss of strength under seismic load conditions and is therefore to be avoided.

11.4.2 *Dimensional limitations*

11.4.2.1 *General*
Dimensional limitations for the joint shall conform to dimensional limitations applicable to all components that it joins. For interior joints 11.4.2.2 or 11.4.2.3 shall also apply. For exterior joints 11.4.2.4 and 11.4.2.5 shall also apply.

C11.4.2.1
To ensure against bond-slip of beam bars and column bars passing through the joint, it is necessary to limit the ratio of column depth, h_c, to beam bar diameter, d_{bb}, and the ratio of beam depth, h_b, to column bar diameter, d_{bc}. Different provisions apply depending on whether or not the adjacent components form potential plastic hinges under seismic attack. The provisions are somewhat less conservative than those given in section 6 for development lengths of bars, reflecting anticipated improved behaviour for development in compression, but a much more conservative approach than that applied to concrete joints has been specified. The result is that column (wall) depths will be large. For example, if the beam is reinforced with D20 bars having f_y = 300 MPa, 11.4.2.2(a) will require a minimum wall depth of 1800 mm. Similarly, beam depth will need to be substantial when walls are subjected to moment reversal through the joint.

11.4.2.2 *Minimum horizontal dimension, h_c*

The minimum horizontal dimensions for the joint, within the plane of loading, h_c, shall be not less than:

(a) Where beam potential plastic hinges may form at column faces:

$h_c = 90\ d_{bb}$ for $f_y = 300$ MPa

$h_c = 150\ d_{bb}$ for $f_y = 500$ MPa

(b) Where beam potential plastic hinges cannot form, or where the critical section is located a distance from the column face not less than h_b:

$h_c = 60\ d_{bb}$ for $f_y = 300$ MPa

$h_c = 100\ d_{bb}$ for $f_y = 500$ MPa

11.4.2.3 *Minimum vertical dimension, h_b*

The minimum vertical dimension for the joint within the plane of loading, h_b, shall be not less than:

(a) Where column potential plastic hinges may form at beam faces:

$h_b = 70\ d_{bc}$ for $f_y = 300$ MPa

$h_b = 130\ d_{bc}$ for $f_y = 500$ MPa

(b) Where columns are not intended to develop potential plastic hinges:

$h_b = 50\ d_{bc}$ for $f_y = 300$ MPa

$h_b = 95\ d_{bc}$ for $f_y = 500$ MPa

These requirements need not be met if it is shown that stresses in external column bars during an earthquake remain in tension or compression over the whole bar length contained within the joint.

11.4.2.4 *Beam bar anchorage of exterior joint*

The overall depth of the column, h_c, at an exterior joint shall be such that effective anchorage is provided to beam reinforcement, commencing at a point one half of the relevant depth of the column or 10 d_{bb}, whichever is the least, from the face at which the beam bar enters the column. Where it can be shown that the critical section of a potential plastic hinge cannot develop in the beam closer than a distance equal to the beam depth or 500 mm from the column face, the development length may be lesser and considered to commence at the column face of entry.

11.4.2.5 *Minimum beam depth at exterior joint, h_b*

The overall depth of the beam, h_b, at an exterior joint shall comply with the dimensional limitations of 11.4.2.3.

11.4.3 *Design forces*

11.4.3.1 *General*

The design forces acting on a beam-column joint core shall be evaluated from the maximum stresses generated by all the components meeting at the joint in equilibrium. The forces shall be those induced when the overstrengths of the beam or beams are developed, except that in cases where a column is permitted to be the weakest component, overstrength of the column is to be taken.

C11.4.3.1
To ensure that a joint possesses adequate reserve strength, the flexural overstrength of the adjacent beams and the corresponding internal forces must be evaluated. The simultaneous forces in the column that maintain joint equilibrium must also be determined. These must correspond to potential plastic hinge regions in the beams that may form either at the column face or at a distance away from the column where the beam overstrengths are developed. The same procedure applies in cases where column hinges are permitted and are expected to form i.e. at top and bottom storeys in ductile framed structures, refer to NZS 3101.

11.4.3.2 *Magnitude of joint forces*
The magnitude of the horizontal shear force, V_{jh}, and the vertical shear force, V_{jv}, in the joint shall be evaluated from a rational analysis taking into account the effect of all forces acting on the joint.

C11.4.3.2
Refer to NZS 3101 for a method for calculating horizontal and vertical joint shear forces.

11.4.4 *Design assumptions*

11.4.4.1 *General*
The design of the shear reinforcement in the joint shall be based on the effective control of a potential failure plane that extends from one corner of the joint to the diagonally opposite corner.

C11.4.4.1
The observed failure plane due to shear in joints of one-way frames bisects the joint along a diagonal from one beam-column edge to another. The reinforcement provided must ensure that the shear force responsible for this failure plane is transmitted without unrestrained yielding of the reinforcement.

11.4.4.2 *Strength reduction factor, ϕ*
In determining the shear strength of the joint the value of the strength reduction factor, ϕ, shall be 1.0.

11.4.4.3 *Maximum permissible horizontal shear stress*
The nominal horizontal shear stress in the joint shall comply with the requirements of 11.3.5.

C11.4.4.3
Nominal joint shear stresses are based on the assumption that the joint width is equal to the column width, b_c. For normal column and beam dimensions this assumption is acceptable. However, if the column width is less than the beam width the result is conservative. In such cases the joint width may be taken as equal to the beam width. Further guidance is provided in NZS 3101.

11.4.4.4 *Shear strength of joint*
The shear strength of the joint shall be assessed as follows:

(a) When potential plastic hinges may develop immediately adjacent to a joint, the entire shear shall be assumed to be resisted by a truss mechanism consisting of horizontal and vertical stirrups or bars and diagonal masonry struts;

(b) When beams are detailed so that potential plastic hinge regions cannot develop immediately adjacent to the joint, a proportion of joint shear resistance may be allocated to the masonry mechanism only, provided that a rational analysis is used or the requirements of 11.4.4.3 and 11.4.5.1 are satisfied.

C11.4.4.4
Refer to NZS 3101 for detailed commentary.

11.4.5 *Horizontal joint shear*

11.4.5.1 *Design shear force, V_{sh}*
The horizontal design shear force to be resisted by the horizontal joint shear reinforcement shall be:

$$V_{sh} = \frac{V_{jh}}{\phi} - V_{mh} \qquad \text{(Eq. 11-10)}$$

C11.4.5.1
The horizontal shear force in the joint may be assumed to be transferred from the level of the top beam flexural reinforcement to the level of the bottom beam flexural reinforcement by diagonal strut action in the masonry core, V_{mh}, and by a truss mechanism, V_{sh}, consisting of diagonal masonry struts, parallel to the potential failure plane, and horizontal stirrup ties and vertical shear reinforcement.

11.4.5.2 *Masonry shear strength, V_{mh}*
The value of V_{mh} shall be assumed to be zero, except:

(a) Where the design precludes the formation of any beam potential plastic hinge regions at a joint; or

(b) Where all beams at the joint are detailed so that the critical section of the potential plastic hinge region is located at a distance from the column face not less than h_b; or

(c) For external joints where the flexural reinforcement is anchored outside the column core in a beam stub, then:

$$V_{mh} = 0.5\ V_{jh} \qquad \text{(Eq. 11-11)}$$

but need not be taken as less than:

$$V_{mh} = v_m b_c h_c \qquad \text{(Eq. 11-12)}$$

where v_m is the grade-dependent masonry shear strength specified in 10.3.2.5.

C11.4.5.2
The background to the reasoning behind the choice of the appropriate value for V_{mh} is given in NZS 3101.

11.4.5.3 *Area of horizontal joint shear reinforcement*
The area of horizontal joint shear reinforcement shall be determined in accordance with 11.3.7.2.

C11.4.5.3
In planar joints, the horizontal joint reinforcement must be adequately anchored around the extreme column vertical reinforcement with a standard 180° hook, or a standard 90° hook with vertical extension sufficient to anchor the reinforcement. The horizontal reinforcement can only be considered effective if it crosses the corner-to-corner potential failure plane.

Intermediate horizontal beam reinforcement passing through the joint between top and bottom beam bars cannot be considered to contribute to horizontal joint shear resistance when beam potential plastic hinge regions can form adjacent to column faces, as in this case the intermediate beam reinforcement will contribute to the overstrength moment of the beam, and hence to joint shear demand.

11.4.5.4 *Spacing of horizontal joint shear reinforcement*
The spacing of horizontal joint shear reinforcement shall not exceed 200 mm.

11.4.6 *Vertical joint shear*

11.4.6.1 *Design shear force, V_{sh}*

The vertical design shear force to be resisted by the vertical joint shear reinforcement shall be:

$$V_{sv} = \frac{V_{jv}}{\phi} - V_{mv} \qquad \text{(Eq. 11-13)}$$

C11.4.6.1

To sustain a diagonal compression field by a truss mechanism, vertical joint shear reinforcement is required. This can be computed in the same way as the horizontal joint shear reinforcement. However, the vertical joint shear force, V_{jv}, may be approximated as follows:

$$V_{jv} = V_{jh}\, h_b / h_c \qquad \text{(Eq. 11-14)}$$

11.4.6.2 *Masonry shear strength, V_{mv}*

The value V_{mv} shall be taken as:

$$V_{mv} = 0.6\, V_{jv} \qquad \text{(Eq. 11-15)}$$

but need not be taken as less than:

$$V_{mv} = v_m b_c h_c \qquad \text{(Eq. 11-16)}$$

where v_m is the grade-dependent masonry shear strength specified in 10.3.2.5.

Where potential plastic hinge regions are expected to form in the column above or below a joint as part of the primary seismic energy dissipating mechanism, V_{mv} shall be assumed to be zero.

C11.4.6.2

The vertical shear force carried by masonry is the same as for elastic joints designed to 11.3, unless the columns are designed for ductility in accordance with provisions of AS/NZS 1170. In such cases, the joint shear transfer is similar to conditions for horizontal joint shear with beam potential plastic hinge regions adjacent to the column, and all shear forces must be carried by specially detailed vertical joint reinforcement.

Where a capacity design process has been adopted, resulting in a high protection against column yielding, intermediate column bars will not be stressed to high levels under design earthquake loading, and the bars can be utilized as effective joint reinforcement for vertical shear transfer through the joint.

11.4.6.3 *Area of vertical shear reinforcement*

The area of vertical joint shear reinforcement shall be determined in accordance with 11.3.8.2.

C11.4.6.3

Where two extreme bars only make up the longitudinal wall reinforcement, intermediate vertical bars, placed between the extreme bars, need to be provided. These need not extend over the full length of a wall but they need to be adequately anchored in the wall above and below the joint. Alternatively, vertical joint stirrups adequately anchored around top and bottom beam reinforcement may be used.

11.4.6.4 *Spacing of vertical shear reinforcement*

The spacing of vertical joint shear reinforcement shall not exceed 200 mm.

12 SECONDARY STRUCTURAL ELEMENTS

12.1 Notation

$C(T_1)$	The ordinate of the elastic site spectrum for the lowest translational period of vibration
R_u	Return period factor for the ultimate limit state (NZS 1170.5)
v_m	Maximum permitted grade-dependent shear stress provided by masonry, defined in 10.3.2.5, MPa
μ	Structural ductility factor, specified in AS/NZS 1170

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C12.1

The following symbols which appear in this clause of the commentary are additional to those used in section 12.

n	*Number of storeys of building*
T	*Period, s*
Δ_u	*Ultimate displacement*
Δ_p	*Plastic displacement*
Δ_y	*Displacement at first yield*
μ_s	*Storey ductility factor*

12.2 Scope

12.2.1

Provisions of this section shall apply to the design of secondary walls, in-fill panels and partitions, as defined in section 2.

C12.2

This section presents provisions which apply for the design of walls which are not considered part of the primary structural system used to support gravity and lateral loads, and covers secondary walls, in-filled panels and partitions. In fact, such walls may be subjected to significant forces and displacements as a result of inertia forces from their self-mass, or as a result of deformations of the primary structural system, particularly under seismic attack. Although detailed numerical design will not generally be necessary for such walls, it is important that detailing provisions are applied that ensure satisfactory performance under the expected response of the structure as a whole to the design loads.

12.3 General principles and requirements

12.3.1

Secondary walls shall be designed to the requirements of 12.4.

12.3.1.1

In-fill panels shall be designed to the requirements of 12.5.

12.3.1.2

Partitions shall be designed to the requirements of 12.6.

12.4 Secondary walls

12.4.1 *General design principles*

12.4.1.1
Structures containing walls in excess of those necessary to carry the seismic lateral loads required by AS/NZS 1170 may be considered to consist of a primary system of walls which supports gravity loads and the entire seismic lateral load; and a secondary system which supports gravity loads and face-loads only.

C12.4.1.1
Some shear-walls do not easily lend themselves to rational analysis under lateral loading, as a consequence of the number, orientation, distribution of openings and complexity of shape of the load-bearing walls. Where such buildings have more walls than are necessary to carry the seismic lateral load, a primary system of walls which is capable of rational analysis may be selected to carry the full seismic lateral load, and the remainder treated as secondary walls. For design purposes, secondary walls are not considered to carry any in-plane lateral loads. However, it is clear that the secondary walls will in fact carry an indeterminate portion of the lateral loads, and must be detailed for the lateral displacements to which they will be subjected. Since secondary walls are still required to support gravity loads, it is essential that only minimal damage occurs to such walls under seismic attack. The provisions of 12.4 are based on this requirement.

12.4.1.2
The classification of the walls of a structure into primary and secondary walls shall be made such that no secondary wall shall have a stiffness greater than one quarter that of the stiffest wall of the primary system, in the relevant direction unless it is established by special study that the secondary wall will not yield at the design level of seismic response.

12.4.1.3
When estimating the design level of seismic load in accordance with AS/NZS 1170 the following provisions shall apply:

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(a) $C(T_1)$ shall be based on the natural period of an estimated composite lateral stiffness of the primary and secondary systems;

(b) The structural ductility factor, μ, shall be determined from the characteristics of the primary system of walls alone.

C12.4.1.2 and C12.4.1.3
The intention of these clauses is to avoid displacements of any wall of the secondary system exceeding its yield displacement. To ensure that satisfactory behaviour is obtained, the primary system should include the stiffest walls, and should be selected in such a manner that the centre of rigidity of the primary system is as near as possible to the centre of rigidity of the complete system of walls.

Since primary walls may be designed for ductility factors of up to $\mu = 4$, *limiting the stiffness of any secondary wall to one quarter that of the stiffest primary wall should result in insignificant inelastic deformation of the secondary wall, unless torsional eccentricity of the system is high. Long, stiff secondary walls may be divided into a series of more flexible walls by the incorporation of separation joints at appropriate centres. In assessing the maximum length of secondary walls to satisfy the limiting stiffness criterion, both flexural and shear deformations should be considered. Where flexure dominates, the length of a secondary wall may be up to one half the length of the longest primary wall for equal wall thickness. Where shear deformations dominate, the secondary wall length should not exceed one quarter of the largest primary element for equal wall thicknesses.*

Figure 12.1(a) and figure 12.1(b) show unsatisfactory and satisfactory subdivision respectively of the walls of a building into primary and secondary systems.

Providing sound engineering judgement is used in the selection of primary and secondary systems, the result will be a conservative design, since the primary system of walls will carry less lateral load than designed for, or will be subjected to lower ductilities than expected for the appropriate structural ductility factor.

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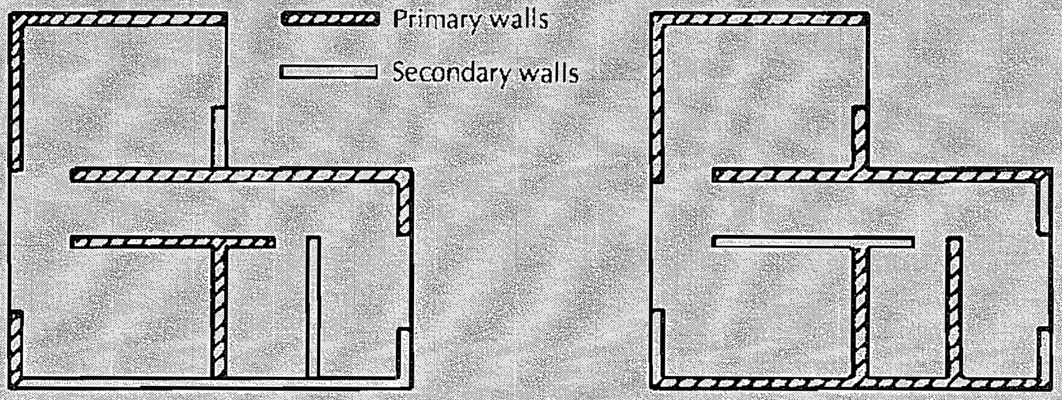
Ignoring the stiffness of secondary walls risks over-estimating the natural period, and an artificially low value for $C(T_1)$. Where the complexity of the secondary walls is such that a reasonable estimate of their stiffness cannot be made, the maximum value of $C(T_1)$, given in AS/NZS 1170, should be adopted.



(a) Unsatisfactory
(Stiffest wall ignored. P & overall systems eccentric)

(b) Satisfactory
(Stiffest walls utilized. P & overall systems have similar centroids)

Figure 12.1 – Subdivision of walls into primary and secondary systems

12.4.1.4

Floor diaphragms shall be designed to span between primary walls under seismic lateral loads.

C12.4.1.4

Since secondary walls cannot be relied upon to provide lateral support for floor diaphragms, the diaphragms must be designed to transmit their inertia forces to primary walls by spanning between them.

12.4.2 *Dimensional limitations*

Secondary walls shall comply with the dimensional limitation requirements of 7.3.3.

12.4.3 *Wall reinforcement*

12.4.3.1

Reinforcement for secondary walls shall comply with the requirements of 7.3.4.

C12.4.3.1

Secondary walls will carry lateral loads in addition to gravity loads. Reinforcement provisions must thus be those for structural walls. However, reinforcement percentages will generally be the minimum permitted, unless governed by gravity load or face-load requirements.

12.4.3.2

The provisions of 7.3.4.7 may only be applied to secondary walls when the primary system of walls is designed to the level of lateral load specified by AS/NZS 1170 and the maximum design shear stress in all primary walls does not exceed the grade-dependent value, v_m specified in table 10.1.

C12.4.3.2

Secondary walls cannot be designed to 7.3.4.7, which allows elastically responding walls to be designed without shear reinforcement provided shear stresses are low, unless the shear stresses in the secondary walls are known to be less than the limits specified in 7.3.4.7. Because of uncertainty in the level of shear in secondary walls, this will only be the case where shear stresses in the primary system satisfy requirements of 7.3.4.7.

12.5 Frames with masonry in-fill

12.5.1 *General design principles*

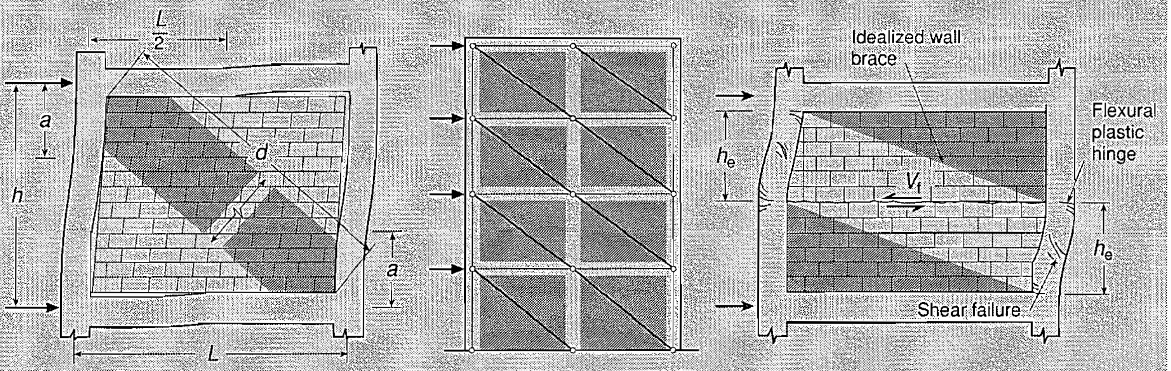
Amd 1 Dec '06 In-fill panels shall be designed to resist all actions resulting from in-plane loads and face loads.

C12.5.1

In-fill panels must be designed to carry the in-plane forces developed as a result of deformation of the in-filled frame under seismic loads, and face loads resulting from wind and earthquake. Seismic face loads are given in NZS 1170.5. However, in assessing panel reinforcement requirements, the in-plane and face-load actions may be considered separately, with the same reinforcement contributing fully in both directions.

Masonry in-fill has been a persistent cause of poor performance of reinforced frames under seismic loading, generally because the in-fill has been considered to be non-structural, and its effect on structural action ignored. The result has often been an induced shear failure of the columns particularly when the masonry does not extend the full storey height. It must be recognized that masonry in-fill panels modify the structural behaviour of the containing frame under lateral load, unless sufficient separation is provided at top and sides to allow free deformation of the frame to occur, in which case the panel must be designed as a partition in accordance with 12.6. It should be noted that even where sufficient separation is provided at top and ends of a panel, the panel will still tend to stiffen the supporting beam considerably, concentrating frame potential plastic hinge regions in short hinge lengths at each end, or forcing migration of hinges into columns, with a breakdown of the weak-beam, strong-column concept. When in-fill panels are constructed without full separation from the frame, the composite action must be considered in analysis and designed accordingly. Structural stiffness is greatly increased, and natural periods reduced. This is significant when determining the appropriate basic seismic coefficient.

For purposes of stiffness and force distribution calculations, the in-filled frame may be represented by an equivalent diagonally braced frame, as shown in figure 12.2 where the effective width of the diagonal masonry strut is one quarter of its length. It is difficult to predict with accuracy the failure load or mode for in-filled frames. Although shear failure of the in-fill is the most common failure mode, other failure modes, including crushing of the diagonal strut, or tension, flexural or shear failure of the columns are possibilities. Failure modes involving shear failure or diagonal compression failure of the in-fill result in rapid strength and stiffness degradation. The resulting increase in period can cause short period structures (t < 0.3 s) to degrade into a period range of higher response. Further, the failure mode invariably results in the formation of a soft storey, with high local ductility demand. Because of these aspects it is necessary to design in-fill panels for high structural type factors unless the panel is constructed as indicated below and is well tied into the frame and reinforced to the extent that a ductile flexural hinging mode, with one column and part of the in-fill panel reinforcement yielding in tension, is assured. In such a case the composite wall will behave as a relatively ductile cantilever wall, and 12.5.1.3 allows it to be designed accordingly. Walls designed on this basis must be carefully supervised and constructed. For full composite action it is essential that close contact between panel and frame be provided at the ends and top. There will generally be considerable difficulty in ensuring close contact at the top, particularly for the grout, unless the top beam is constructed after the panel is built and grouted.



(a) Deformation under shear load

(b) Equivalent braced frame for 2 bay, 4 storey in-filled wall

(c) Knee braced frame concept for sliding shear failure

Figure 12.2 – Equivalent diagonal bracing action of masonry in-fill

12.5.1.1

Except as provided by 12.5.1.2 masonry in-filled frames shall be designed as elastically responding structures, unless a special study is carried out to determine the available structural ductility, μ.

C12.5.1.1

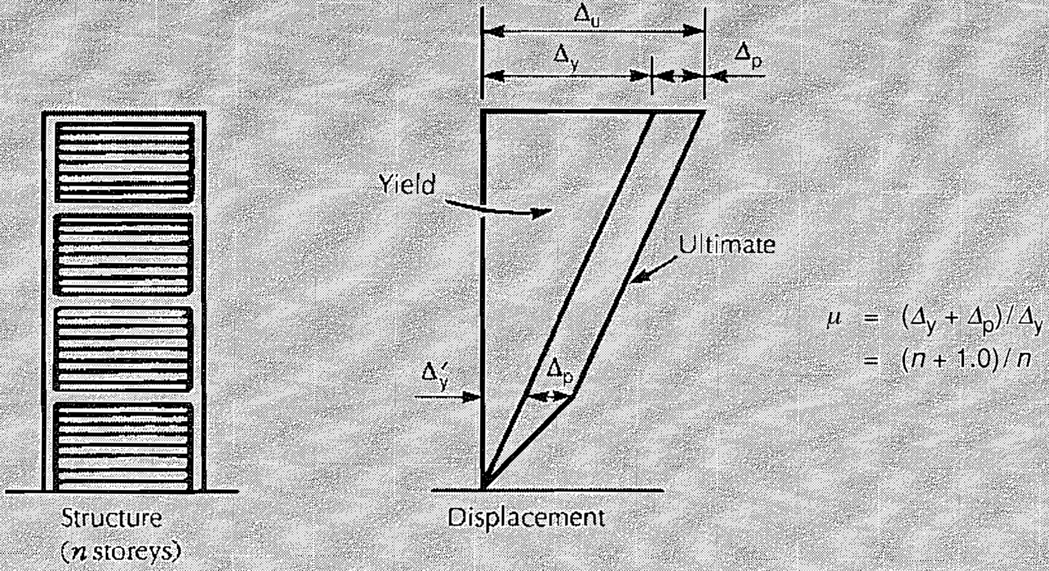
The provisions of this clause are intended to ensure that ductility demand will be small at any level where shear failure of the in-fill can occur. However, it is accepted that this is conservative for regular structures of only a few storeys. Consequently, a rational analysis may be adopted to determine a structural ductility factor that results in a component displacement ductility demand at any level no greater than that implied for structures of limited ductility (that is $\mu = 2$ in accordance with AS/NZS 1170). For regular structures of constant storey height, this results in the following expression for the maximum value of μ:

$$\mu = \frac{n+1}{n} \quad \text{(Eq.12-1)}$$

where n is the number of storeys.

Equation 12-1 is based on an assumed linear deflection profile at yield with all inelastic displacement occurring at one level as illustrated in figure 12.3.

Single storey masonry in-fill may be designed in 140 mm blockwork, provided the in-filled element will behave as an elastically responding element under the design level earthquake, and provided shear stress limitations of 7.3.4.7 are satisfied.



$$\mu = (\Delta_y + \Delta_p)/\Delta_y$$
$$= (n + 1.0)/n$$

Figure 12.3 – Ductility of in-fill frame based on maximum storey ductility of $\mu_S = {}^4/_2 = 2.0$

12.5.1.2

Masonry in-filled frames designed and detailed to ensure that in-fill and frame act together in full composite action as a shear wall shall be designed as shear walls to the requirements of section 7, and to appropriate structural ductility factors of section 3. Amd 1 Dec '06

12.5.1.3

In-fill panels with openings shall be subject to special study to ensure diagonal bracing action can be obtained, and to investigate the effects of structural modification caused by the openings.

C12.5.1.3

Openings in in-fill panels tend to destroy the diagonal bracing action of the in-fill, and cause modification of the structural action, often with premature shear failure of the in-fill. Consequently, openings should be avoided unless detailed studies are carried out to adequately define the modified behaviour, and to enable a rational design to be obtained. Minimum reinforcement requirements are the same as for structural walls. See also 7.4.8 for openings.

12.5.1.4

Structural in-fill panel reinforcement shall be connected to adjacent beams and columns by lapped starter bars, or by welding, or by other approved means, to ensure that composite action results.

12.5.1.5

In-fill panels separated from the structural system such that the ultimate limit state inter-storey deflections calculated in accordance with NZS 1170.5 are accommodated shall be considered to be partitions and shall comply with the requirements of 12.6. Amd 1 Dec '06

Table A1 – Maximum concrete masonry stresses and steel stress range for the design of sections at the serviceability limit state

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Stress case	Load category			
	I	II	III	IV
	Immediately after transfer before time dependent losses	Permanent loads plus imposed loads of long duration, or permanent loads plus frequently repetitive loads	Specified service loads for buildings where load category II does not apply	Permanent loads plus infrequent combinations of imposed loads
Uncracked section: compression	$0.6\ f'_{\mathrm{mi}}$	$0.4\ f'_{\mathrm{m}}$	$0.45\ f'_{\mathrm{m}}$	$0.55\ f'_{\mathrm{m}}$
Tension across construction joints	zero	zero	zero	zero
Tension in monolithic concrete masonry	$0.5\sqrt{f'_{\mathrm{mi}}}$	zero	$0.5\sqrt{f'_{\mathrm{m}}}$	$0.5\sqrt{f'_{\mathrm{m}}}$
Cracked section: compression	$0.6\ f'_{\mathrm{mi}}$	$0.4\ f'_{\mathrm{m}}$	$0.45\ f'_{\mathrm{m}}$	$0.55\ f'_{\mathrm{m}}$
Steel stress range (MPa)	200	100	200	200

NOTE – For stress combinations which include differential temperature effects the maximum permitted compression stress for both cracked and uncracked sections may be increased to the smaller of:

(a) $0.75\ f'_{\mathrm{m}}$; or

(b) The value listed in table A1 plus ${}^{2}/_{3}\ \alpha_{\mathrm{m}} E_{\mathrm{m}} T$.

A3.1.3 *Design for deformations*

Provision shall be made for the effects on parts of the structure or adjoining structure of elastic and plastic deformation. The effects of temperature, creep, and shrinkage shall be considered.

A3.1.4 *Consideration of buckling*

The possibility of buckling in a component between points where the concrete masonry and the prestressing steel are in contact and of buckling in thin webs and flanges shall be considered.

CA3.1.4

This refers to the type of post-tensioning where the tendon makes contact with the prestressed concrete member intermittently. Precautions should be taken to prevent buckling of such members. In particular, if thin webs or flanges are under high pre-compression, buckling is possible between supports of slender members. If the tendon is in complete contact with the member being prestressed, or is an unbonded tendon in a duct not excessively larger than the tendon, buckling of the member when the prestressing force is introduced is not possible.

A3.1.5 *Loss of area due to ducts*
In calculations of section properties prior to bonding of tendons, the effect of loss of area due to open ducts shall be considered. In pre-tensioned components and in post-tensioned components after grouting, section properties may be based on gross sections, net sections, or effective sections using transformed areas of bonded tendons and reinforcing steel.

A3.1.6 *Non-straight tendons*
Where tendons are subjected to deviations from a straight line, allowances shall be made for the forces causing those deviations.

CA3.1.6
The deviation of cables from a straight line causes forces which may result in damage if there is inadequate cover or resistance.

A3.1.7 *Stress concentration*
Stress concentrations due to prestressing shall be considered in the design.

CA3.1.7
Stress concentrations which can lead to cracking can arise where inserts or ducts are formed in prestressed members. Stress concentrations also arise in anchorage zones of prestressing cables (see A3.8).

A3.2 Basic assumptions

A3.2.1 *Ultimate limit state*
In designing for strength at the ultimate limit state, the assumptions provided in 10.2.2 shall apply.

A3.2.2 *Serviceability limit state*
In investigating sections at the serviceability limit state, after transfer of prestress and at cracking load, elastic theory shall be used with the assumptions that at any section where the permissible concrete masonry tensile stresses are exceeded the section shall be assumed to be cracked and to have no tension capacity in that part of the section. The effect of tension stiffening between cracks may be included.

A3.3 Unbonded tendons

Where unbonded tendons are considered:

(a) The use of unbonded tendons is permitted provided they are in accordance with NZS 3109 and the exposure classification as defined in table 4.1 is not *C* or *U*;

(b) Ultimate design flexural strength shall be computed in accordance with A3.6.

CA3.3
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To avoid a sudden collapse of unbonded prestressed concrete masonry structures under extreme imposed actions and especially seismic actions, the minimum value of average prestress in rectangular sections must always be substantially higher than the modulus of rupture. The calculation of the tensile stress at cracking is based on the assumption that the stress distribution is linear.

In a structure with unbonded tendons, tendon failure may result in the release of the prestressing force along the whole length of the tendons. Such an event could lead to the collapse of the whole structure. Consideration should be given to the consequence of such a failure to the overall stability of the structural system. One consideration would be to use reduced tendon lengths between anchorages or tendon couplers capable of acting as intermediate anchorages.

A3.4 Maximum stresses in prestressing steel

Tensile stress in prestressing tendons shall not exceed the following:

(a) Due to jacking force .... 0.94 f_{py}
but not greater than the lesser of 0.80 f_{pu} or the maximum value recommended by the manufacturer of prestressing tendons or anchorages;

(b) Immediately after prestress transfer .... 0.82 f_{py}
but not greater than 0.74 f_{pu};

(c) Post-tensioning tendons, at anchorages and couplers, immediately after tendon anchorage .... 0.70 f_{pu}

The requirements of table A1 shall also be satisfied at the serviceability limit state.

CA3.4

Maximum stresses in tendons recognize the higher yield strength of low-relaxation wire and strand meeting the requirements of ASTM A421 and A416. Refer to NZS 3101 for further details relating criteria between f_{py} and f_{pu}.

Designers should be concerned with setting a limit on final stress when the structure is subject to corrosive conditions or repeated loadings.

A3.5 Loss of prestress

A3.5.1 *Sources of prestress loss*

To determine the effective prestress, allowance for the following sources of loss of prestress shall be considered:

(a) Anchorage seating;

(b) Elastic shortening of concrete masonry;

(c) Creep of concrete masonry;

(d) Shrinkage of concrete masonry;

(e) Relaxation of steel stress;

(f) Frictional loss due to intended or unintended curvature in the tendons.

CA3.5.1

Prestress losses may be expected to vary substantially for different applications. Although the actual loss will have little effect on the design strength of the member, it will affect serviceability limit state stresses and behaviour, such as deflection, camber and cracking load. These aspects can control the design. Methods of computing losses are given in References A1, A2 and A3. Reference A7 reports research conducted using NZ pumice aggregate masonry blocks which supported the use of a creep coefficient of 3.0.

AS 3700 quotes a creep coefficient of 2.5 and BS 5628 quotes a creep coefficient value of 3.0. For masonry blocks using aggregate that is distinctly more dense than pumice, a creep coefficient corresponding to that in AS 3700 is recommended.

A3.5.2 *Friction losses when post-tensioning*

Friction losses in post-tensioned tendons shall be based on experimentally determined wobble and curvature coefficients, and shall be verified during stressing operations. The values of coefficients assumed for design, and the acceptable ranges of jacking forces and steel elongations shall be defined. These friction losses shall be calculated as follows:

$$P_x = P_s e^{-(KL + \mu\alpha)} \qquad \text{(Eq. A-1)}$$

Where $(KL + \mu\alpha)$ is not greater than 0.3, Equation A-2 may be used:

$$P_x = \frac{P_s}{(1 + KL + \mu\alpha)} \qquad \text{(Eq. A-2)}$$

CA3.5.2

Refer to NZS 3101 for a typical range of friction coefficients. Due to the many types of ducts, tendons and wrapping materials available, these values can serve only as a guide. Guidance on the friction that can be expected with particular type tendons and particular type ducts can be obtained from the specialist prestressing contractors. An unrealistically low evaluation of the friction loss can lead to improper camber of the structure and inadequate prestress. Overestimation of the friction may result in extra prestressing force if the estimated friction values are not attained in the field. This could lead to excessive camber and excessive shortening of a member. If the estimated friction factors are determined to be less than those assumed in the design, the stressing force should be adjusted to give only that theoretical prestressing force in the critical portions of the structure required by the design.

A3.5.3 *Friction losses when pre-tensioning*

Friction losses in deflected pre-tensioned tendons shall be based on field measured values or on values derived from similar forces, geometric profiles and tendon deflection hardware.

A3.5.4 *Connection losses*

Where prestress in a component may be reduced through its connection with adjoining elements or contact with the ground, such reduction shall be allowed for in the design.

A3.6 Flexural strength

A3.6.1 *Design strength*

The design strength of components containing prestressed reinforcement shall be taken as the nominal strength times the strength reduction factors, given in 3.4.7.

CA3.6.1

The computation of ultimate flexural strength may be carried out using the same equations as those provided in the ACI 318 Code.

(a) Rectangular sections, or flanged sections in which the neutral axis lies within the flange:

$$M^* = \phi\left[A_{ps} f_{ps}\left(d - \frac{a}{2}\right)\right] \qquad \text{(Eq. A-3)}$$

where a is the depth of the equivalent rectangular stress block as defined by 10.2.2.6.

(b) Flanged sections in which the neutral axis falls outside the flange:

$$M^* = \phi\left[A_{pw} f_{ps}\left(d - \frac{a}{2}\right) + 0.85 f'_m (b - b_w) h_f \left(d - \frac{a}{2}\right)\right] \quad \text{(Eq. A-4)}$$

where

$A_{pw} = A_{ps} - A_{pf}$

and

$A_{pf} = 0.85\,(b - b_w)\,h_f / f_{ps}$

A_{pf} and A_{pw} are those portions of area of the prestressing steel required to develop the compressive strengths of the overhanging flanges and the web respectively, where h_f is the flange thickness.

Development and full explanations of these equations are contained in Reference A4.

A3.6.2 *Stress in tendons*

The nominal strength shall be determined from basic assumptions in 10.2.2 with allowance being made for the additional strain in prestressed reinforcement due to prestressing. The stress in the prestressing tendons shall be determined in accordance with A3.6.3, or alternatively where appropriate, it may be determined by the method given in A3.6.4.

CA3.6.2

The strain in a prestressing tendon is assumed to be equal to the strain calculated from the assumption of plane sections remaining plane plus the strain due to prestressing of f_{se}/E_{sp}.

A3.6.3 *Compatability method for determining prestress*

The stress in prestressed reinforcement in all cases may be determined from strain compatibility and an appropriate stress-strain relationship for the prestressing tendons. In calculating the strain in the prestressing tendons allowance shall be made for strains imposed by prestressing.

CA3.6.3

This method of determining the stress in a prestressing tendon is the most general method, which can be applied in all situations. An appropriate stress-strain relationship may be found from suppliers of prestressing tendons or from literature such as Reference A5.

A3.6.4 *Approximate method for determining prestress*

In lieu of the method defined in A3.6.3 the stress in prestressing tendons located in regions of flexural cracks may be determined as set out in this section.

CA3.6.4

The stress in prestress tendons at the design limit state is influenced by the geometry and deformation of the component and the nature of tendon bonding. For bonded tendons and for unbonded tendons of beams and columns, the expressions given in A3.6.4.1, A3.6.4.2 and A3.6.4.3 are adaptations of those in NZS 3101. Studies on unbonded prestressed masonry walls reported in reference A-6 have shown Equation A-8 to be more accurate for this component type.

A3.6.4.1 *Tendon stress when using bonded tendons*

For components using bonded prestress tendons and subject to axial design forces of less than 0.5 $A_g f'_m$ the effect of axial load may be neglected and the tendon stress may be determined from:

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$$f_{ps} = f_{pu}\left(1 - \frac{\gamma_p}{0.85}\left[p_p \frac{f_{pu}}{f'_m} + \frac{d}{d_p}(\omega - \omega')\right]\right) \quad \text{(Eq. A-5)}$$

If any compression reinforcement is taken into account when calculating f_{ps} by Equation A-5, the term

$$\left[p_p \frac{f_{pu}}{f'_m} + \frac{d}{d_p}(\omega - \omega')\right]$$

shall be taken not less than 0.17 and d shall be taken not greater than 0.15 d_p.

Where the design axial forces are equal to or greater than 0.5 $A_g f'_m$ the component is to be designed as a compression component in accordance with A3.1.2.

A3.6.4.2 *Tendon stress in unbonded beams and in unbonded columns*
(a) Where the span-to-depth ratio is 35 or less:

$$f_{ps} = f_{se} + 70 + \frac{f'_m}{100 p_p} \quad \text{(Eq. A-6)}$$

but f_{ps} in Equation A-6 shall be taken not greater than f_{py} nor (f_{se} + 400).

(b) Where the span-to-depth ratio is greater than 35:

$$f_{ps} = f_{se} + 70 + \frac{f'_m}{300 p_p} \quad \text{(Eq. A-7)}$$

but f_{ps} in Equation A-7 shall be taken not greater than f_{py} nor (f_{se} + 200).

A3.6.4.3 *Tendon stress in unbonded walls*
For masonry walls with multiple tendons the stress in each tendon shall be calculated separately using:

$$f_{ps} = f_{se} + \frac{20}{L_{ut}} \frac{f'_m}{f_m} \frac{h_e}{L_w}\left[d_i - 1.4 \frac{f_m}{f'_m} L_w\right] \quad \text{(Eq. A-8)}$$

but f_{ps} in Equation A-8 shall be taken not greater than f_{py}.

CA3.6.4.3
Equation A-8 has been derived using the masonry stress-strain properties specified in 3.4.3 and 10.2.2.6 when using no confining plates and using the modulus of elasticity of reinforcement specified in 3.4.4. Reference A-6 may be consulted for adaptation of Equation A-8 when using materials having other material characteristics.

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A3.6.5 Stress in non-prestressed reinforcement
The stresses in non-prestressed reinforcement, including deformed bars or unstressed strands, shall be determined from strain compatibility.

CA3.6.5
Unstressed prestressing tendons, in the form of strands, should not be relied upon to act as compression reinforcement.

A3.7 Limits for reinforcement in flexural components

A3.7.1 *Strength to exceed cracking moment*
The design moment in flexure for any section at the ultimate limit state shall be not less than 1.2 times the cracking moment computed on the basis of a modulus of rupture of $0.5\sqrt{f'_m}$, except for flexural components with shear and flexural strengths at least twice that required to meet the ultimate limit state requirements of AS/NZS 1170.

CA3.7.1
This provision is a precaution against abrupt flexural failure resulting from rupture of the prestressing tendon immediately after cracking. Typical flexural members require considerable additional load beyond cracking to reach ultimate capacity. Thus, considerable deflection warns that the ultimate capacity is being approached. However, if the ultimate capacity develops shortly after cracking, warning increased by deflection may not occur.

A3.8 End regions

CA3.8
Many design methods for end regions have been developed and they can lead to widely differing answers. For detailed design techniques that are thought to be appropriately conservative, see Reference A3.

A3.8.1 *Design forces*
Reinforcement shall be provided when required in the anchorage zone to resist bursting, splitting, and spalling forces induced by the tendon anchorage. Regions of abrupt change in section shall be adequately reinforced. Refer to 10.2.2.8.

A3.8.2 *Bearing*
End blocks shall be provided when required for end bearing or for distribution of concentrated prestressing forces. Anchorage forces shall not result in concrete masonry bearing stresses greater than specified in 10.2.2.8.

A3.8.3 *Maximum jacking load*
Post-tensioning anchorages shall be designed to support the maximum jacking load at the concrete masonry strength at the time of prestressing, and the end anchorage region shall be designed to develop at least 95 % of the guaranteed ultimate tensile strength of the tendons, or the calculated tensile force, whichever is the greater with a value of $\phi = 0.85$.

A3.9 Shear strength

The requirements of 10.3 shall be satisfied.

A3.10 Serviceability limit state

CA3.10
Crack widths, deflections and performance under frequently repetitive loads must be within acceptable limits. Maximum stress limits of A3.10.1 should control behaviour at the serviceability limit state. They do not necessarily guarantee adequate structural capacity at the ultimate limit state which should be checked in conformance with other requirements of the Standard. Table A1 gives 4 different load categories which should cover all cases. In general, load category III as described in table A1 will be used for buildings except where imposed loads may be of long duration. A long duration load may be considered as one which is present for 3 months or more.

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Lower stress limits are specified for load category II in recognition of the fact that loads which are of a long duration or frequently repetitive may cause increased deflections or increased crack widths, or both. In general frequently repetitive loads may be considered as those which will occur more than 50 000 times in the life of the structure. Building loads will not normally be considered as frequently repetitive loads. For a more complete discussion on fatigue loading see Reference A6.

A3.10.1 *General*
In general the requirement for adequate performance at the serviceability limit state will be achieved either by designing on the basis of homogeneous or uncracked sections or by performing an analysis based on cracked sections in accordance with A3.2.2:

(a) Where uncracked sections are considered, the flexural concrete masonry stresses shall be not more than the appropriate values in table A1;

(b) Where analysis is based on cracked sections in accordance with the principles of A3.2, bonded reinforcement (whether non-prestressed or prestressed) must be present. For this case the compressive stresses in the concrete masonry and the range of stress (that is the total maximum variation from compression to tension for non-prestressed steel and the increase in tension for prestressing steel) in the bonded steel shall be not more than the appropriate values in table A1. In addition the maximum tension in the prestressing steel at transfer and in the final service condition after all design losses shall not exceed 0.70 f_{pu}. Allowance shall be made for the effects of creep and shrinkage in the concrete masonry on the stresses in the reinforcement and the concrete masonry.

(c) The exception for which the stress limits of table A1 does not apply, and special studies are mandatory, are structures that have an exposure classification of *C* or *U* as defined in table 4.1.

CA3.10.1
High localized differential temperature compression stresses are concentrated close to exposed surfaces. They are very shallow and therefore the allowable compression stress may be increased as given in the note to table A1.

CA3.10.1(b)
This clause gives simple rules for the design of cracked prestressed concrete masonry also known as "partially prestressed" concrete masonry. Where the reduction in moment of inertia can be large at cracking, the loss of stiffness and increase in deflection is large. For this reason where cracking is permitted, the designer may need to compute the deflection based on cracked cross-sections and the transformed areas of bonded steel.

Consult NZS 3101 for selection of the steel stress range limits.

A3.10.2 *Deflections*
For calculations of deflection it is permissible to assume that the section is uncracked provided the concrete masonry tensile flexural stress is less than $0.3\sqrt{f'_m}$ under sustained loading, or less than $0.4\sqrt{f'_m}$ under short-term loading.

A3.10.3 *Frequently repetitive loads*
For components subject to frequently repetitive loads the possibility of inclined diagonal tension cracks forming under stresses lower than those induced by monotonic loading at the serviceability limit state shall be taken into account.

A3.10.4 *Prestress variation*
In all cases where curvature occurs in post tensioning tendons, allowance shall be made for variation of the tendon force at any point by ±5 % from the calculated values.

REFERENCES

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A2 Naamon, A.E., and Siriak Soren, A. "Serviceability Based Design of Partially Prestressed Beams". PCI Journal Vol. 24, No. 3, May – June 1979.

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APPENDIX B
DETERMINATION OF CONCRETE MASONRY COMPRESSIVE STRENGTHS

(Normative)

B1 From component strengths of grout and masonry units

B1.1 Notation

f_{cb}	Mean strength of concrete masonry unit MPa,
f_g	Mean grout strength, MPa,
f_m	Mean masonry compressive strength, MPa,
f'_{cb}	Characteristic strength of masonry unit, MPa,
f'_g	Characteristic grout strength, MPa,
f'_m	Characteristic masonry compressive strength, MPa,
x_{cb}	Standard deviation of strength of concrete masonry unit MPa,
x_g	Standard deviation of strength of grout, MPa,
x_m	Standard deviation of strength of masonry strength, MPa,
α	Maximum ratio of net area to gross area of masonry unit

CB1.1

A Gross area of prism

B1.2 Scope

B1.2.1

This Appendix specifies an alternative method to the prism test method of Appendix B2, for defining the design compressive strength of masonry, f'_m, for strengths greater than 12 MPa.

CB1.2.1

The procedure specified in Appendix B has been adapted from that developed by Priestley and Chai[B1] *and specified in NZS 4230:1990 to account for f'_m being a characteristic strength conforming with Equation B-3. This has resulted in a modification to an equation, which previously contained a 0.75 multiplier to represent a lower-bound value. The approach adopted here accounts for the fact that the strength of concrete masonry units and the grout strength are independent normally distributed variables.*

It should be noted that mortar strength is not included because in the theoretical considerations the influence of mortar strength is very low.

If the grout used incorporates an expansive additive, it is likely that Equation B-1 will be further conservative, and the advantages of using Appendix B2 to assess f'_m will be more apparent.

The method given in this Appendix will be particularly useful in the design stage in assessing a suitable design value for f'_m, since typical local values for f'_{cb} , f_{cb} , x_{cb} and f'_g , f_g , x_g will be readily available from masonry manufacturers and grout suppliers. If data is not readily available then testing based *on 10 samples of each product should be used to generate the data. It should be noted that substitution of minimum allowed values for f_{cb}, f_g, x_{cb} and x_g into the equations represents the minimum default values for f'_{cb} of 12.5 MPa at f'_g of 17.5 MPa and results in a design value of close to the f'_m = 12 MPa (12.9 MPa) designated as the default value with Type B Observation.*

B1.3 Masonry compressive strength

B1.3.1

Characteristic masonry compressive strength, f'_m can be calculated from the strengths of the grout and the masonry unit using the following equations:

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$$f_m = 0.59\ \alpha f_{cb} + 0.90(1-\alpha)f_g \quad \text{Eq. B-1}$$

$$x_m = \sqrt{0.35\alpha^2 x_{cb}^2 + 0.81(1-\alpha)^2 x_g^2} \quad \text{Eq. B-2}$$

$$f'_m = f_m - 1.65\ x_m \quad \text{Eq. B-3}$$

CB1.3.1

The value of α (net area to gross area ratio) used in Equations B-1 and B-2 should be the maximum for the characteristic block type used in the masonry construction. For example, if the wall is constructed mainly of open bond beam units, α should be based on the area of face-shells (see AS/NZS 4455) plus the area of two cross webs, even though the net block area at the top of the unit consists of the two face-shells only.

B1.3.2

Mean masonry unit strength f_{cb} used in Equation B-1 shall be established by testing in accordance with AS/NZS 4456. The standard deviation x_{cb} shall be calculated in accordance with AS/NZS 4456.

B1.3.3

Mean grout strength shall be established by testing based on a minimum number of 10 samples in accordance with NZS 3112: Part 2. The standard deviation x_g shall be calculated in accordance with NZS 3112.

B2 From compression testing of composite masonry prisms

CB2

Design Engineers who wish to specify a compressive strength, f'_m greater than 12 MPa but who do not have information which will enable them to confidently choose a design value for f'_m are advised to carry out sets of prism tests at the design stage.

It is suggested that a minimum of 2 sets of prisms be made by the masonry contractors to be employed on the job from masonry materials which are to be specified for the work. The results of the tests carried out at this stage cannot be taken as a guarantee of results likely to be obtained from testing during construction and should therefore be conservative. The result of any one test should not be less than 1.25 f'_m and the average should approach 1.5 f'_m.

It is recommended that prisms be constructed on the job using actual materials delivered for the job at an early stage so that testing can be carried out if possible before construction of critical elements.

If an expanding admixture is to be used then it is important to use infill grout appropriately dosed. It is NOT possible to then test this grout for strength after dosing using standard steel cylinder moulds.

B2.1 Scope

B2.1.1

This Appendix specifies the procedure for the making and testing of masonry prisms for the determination of compressive strength of masonry where a design engineer uses compressive stresses greater than 12 MPa.

B2.2 Number and size of prisms

B2.2.1

Each test shall consist of a set of three prisms except as provided in B2.3.2.

B2.2.2

The thickness of a prism shall be the same as the thickness of the element represented in the structure.

B2.2.3

The length of a prism shall be equal to or greater than the thickness and not less than the length of one masonry unit.

B2.2.4

The height of a prism shall be a minimum of three courses and not less than three times the thickness of the prism.

B2.3 Prism construction and storage

B2.3.1

Prisms shall be made from the same materials as those used in the element or elements represented and shall be constructed in the same bond as the structure except that where a prism is only one unit in length, stack bonding shall be permitted. No reinforcement shall be included.

B2.3.2

Each mason engaged in constructing the elements represented by a test shall lay up at least one prism of the test set. Where more than three masons are so engaged, the number of prisms comprising the test set shall be increased.

B2.3.3

Workmanship shall be that used in the construction and in particular the following features shall be the same as those used in the elements represented:

(a) The condition of the masonry units;

(b) The mortar bedding;

(c) The thickness and tooling of joints;

(d) The grouting.

B2.3.4

Prisms shall be constructed on a rigid, level base in the position in which they are to be stored until removal for testing.

B2.3.5

Immediately after laying up, the prisms shall be covered to reduce evaporation and to prevent exposure to dripping or running water and to protect them from direct sunlight. They shall be left undisturbed until grouted.

B2.3.6

Grouting shall be carried out at the same time as in the elements represented. Grout shall be placed in a single lift and compacted by the same means as employed in the elements represented. After compaction, the top surfaces shall be finished smooth and level by trowelling.

B2.3.7

Immediately after grouting, the prisms shall be covered and stored in the conditions as required in B2.3.5 for a period of not less than five days before being transported to the test laboratory.

B2.3.8

Care shall be taken in transporting the prisms to the test laboratory to avoid any damage. They shall rest on a bed of sand or sawdust or other soft material and shall be restrained from movement.

B2.3.9

Prisms shall be stored at the laboratory at a temperature not exceeding 25 °C until they are tested which shall be not later than 28 days after the date of grouting.

CB2.3.9

Because the compression testing of masonry prisms is intended to provide confirmation of the design strength of masonry rather than to be a means of quality control, maximum storage temperature in the laboratory and maximum age of testing only are specified. In recognition that it may not always be practicable to closely control curing conditions, uncontrolled conditions within the maximum limits specified are acceptable. It should be borne in mind that where curing conditions are uncontrolled some reduction in strength should be expected.

B2.4 Determination of compressive strength of masonry prisms

B2.4.1 *Apparatus*

B2.4.1.1

The testing machine shall comply with the requirements of NZS 3112: Part 2, section 6 except that the upper platen may be rigidly seated, in which case the test procedure of B2.4.2.2 shall be followed.

CB2.4.1.1

In order to be able to verify that the requirements are met, the load capacity of the testing machine should not be less than 1.75 A f'_m *where A is the gross area of the prism. Where prisms are being tested at the design stage to establish an appropriate value for* f'_m *then the capacity should not be less than 2.0 A* f'_m.

B2.4.1.2

When the bearing areas of the platens are not sufficient to cover the ends of the masonry prisms, auxiliary steel bearing plates at least 20 mm larger than the prism dimensions shall be placed between the platens and the masonry prism. Such plates shall have a thickness equal to at least one third of the distance from the edge of the platen to the most distant corner of the prism, but in no case less than 15 mm. The surfaces of the steel plates shall not depart from a plane by more than 0.25 mm in any 150 mm dimension.

B2.4.2 *Test procedure*

B2.4.2.1

The determination of prism compressive strength using a test machine having a spherically seated upper platen shall be made in the following manner:

(a) Examine the prism and report any damage or visual defects;

(b) Record dates of laying up, grouting, and testing;

(c) Record the width and length of the prism by measuring the sides at the central section, to within 1 mm;

(d) Wipe the bearing surfaces of the platens and auxiliary bearing plates and check that the ends of the prism are clean and dry;

(e) Cut two pieces of 12 mm wood fibre softboard 20 mm larger than the prism dimensions and place one piece centrally on the lower platen;

(f) Locate the prism centrally on the softboard on the lower platen;

(g) Place the second piece of softboard centrally on top of the prism;

(h) Check that the load indicator is at zero;

(i) Carefully bring the upper platen in contact with the upper piece of softboard to ensure uniform seating;

(j) Apply the load continuously and without shock at a constant rate of between 1 and 10 MPa/min;

(k) Record the maximum load, in kilonewtons, carried by the specimen during the test;

(l) Calculate the compressive strength of the prism by dividing the maximum load by the gross cross-sectional area;

(m) Report the compressive strength of the prism to the nearest 0.1 MPa.

B2.4.2.2

The determination of prism compressive strength using a test machine having a rigidly seated upper platen shall be made in the following manner:

(a) Use steps (a) to (e) of B2.4.2.1;

(b) Trowel a layer of neat high strength gypsum plaster on to the top surface of the prism;

(c) Locate the prism centrally on the softboard on the lower platen;

(d) Attach a sheet of paper to the underside of the top platen or auxiliary bearing plate if used and bring into contact with the plaster on the top of the prism to ensure bearing over the whole area;

(e) Leave the prism undisturbed in the testing machine until the plaster has attained sufficient strength for the test;

(f) Separate the upper platen from the prism and place the second piece of softboard centrally on top of the prism;

(g) Proceed with the test as in B2.4.2.1 steps (h) to (m).

REFERENCES

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APPENDIX C
BOLTED CONNECTIONS IN MASONRY

(Normative)

C1

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This Appendix gives the design strengths for bolts embedded in masonry.

C2

Where necessary bolts shall be protected from corrosion to comply with the requirements of section 4 Design for Durability.

C3

Bolts shall be embedded in mortar, grout or other suitable materials from the surface of the masonry for the full length of the bolt into the masonry.

C4

Bolts shall be provided with a positive means of preventing loosening through tightening or loading either by cranking through their mid-length, forming an L or J shape or by welding suitable plates or bars to their heads.

C5

Detailing of bolts shall take into account:

(a) Impact loads;

(b) Vibratory loads;

(c) Effect of volumetric changes due to shrinkage, creep and temperature.

C6

Amd 1 Dec '06

The design strength in shear or tension of bolts embedded in observation types A, B or C reinforced masonry shall be limited to the values given in table C1.

C7

The minimum edge distance for bolts measured to the centre of the bolt shall be not less than the required embedment length and the minimum spacing between bolts shall be not less than twice the embedment depth except where:

(a) The load is reduced in the same proportion as the edge distance or spacing is reduced;

(b) The bolts are confined by reinforcing, then the edge distance may be reduced by 50 % but shall be not less than the required cover.

C8

Cast in bolts subjected to combined shear and tension loading shall be designed to satisfy:

$$\frac{N_t^*}{\phi N_t} + \frac{V_f^*}{\phi V_f} \leq 1.2 \qquad \text{(Eq. C-1)}$$

where

N_t^* is the design axial tension load

ϕN_t is the axial tension strength

V_f^* is the design shear load

ϕV_f is the shear strength, refer to table C1

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Table C1 – Design strength in shear and tension of bolts cast into reinforced masonry

Diameter of bolt (mm)	**Embedment** (mm)	**Observation Type A and B masonry** ϕN_t **and** ϕV_f (kN)	**Observation Type C masonry** ϕN_t **and** ϕV_f (kN)
12	100	10	5
16	125	15	8
20	150	25	13
24	175	35	18

C9 Shear on bolted connections in masonry

C9.1

Where bolts are embedded in a grouted flue or cavity and protrude horizontally from the face of the units, the bolt shall be bedded right to the face of the unit. This may necessitate using a significantly oversized hole through the face of the shell of the unit or the skin (see figure C1).

C9.2

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Design strengths are not given for proprietary fixings as these vary considerably with type and depth of fixing and shell-face thickness.

C9.3

Edge distance requirements are illustrated in figure C2.

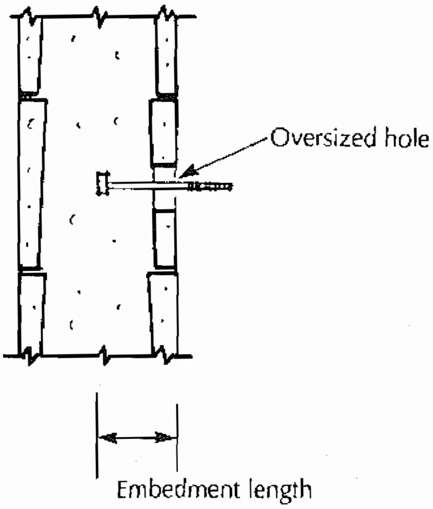

Figure C1 – Typical bolt detail

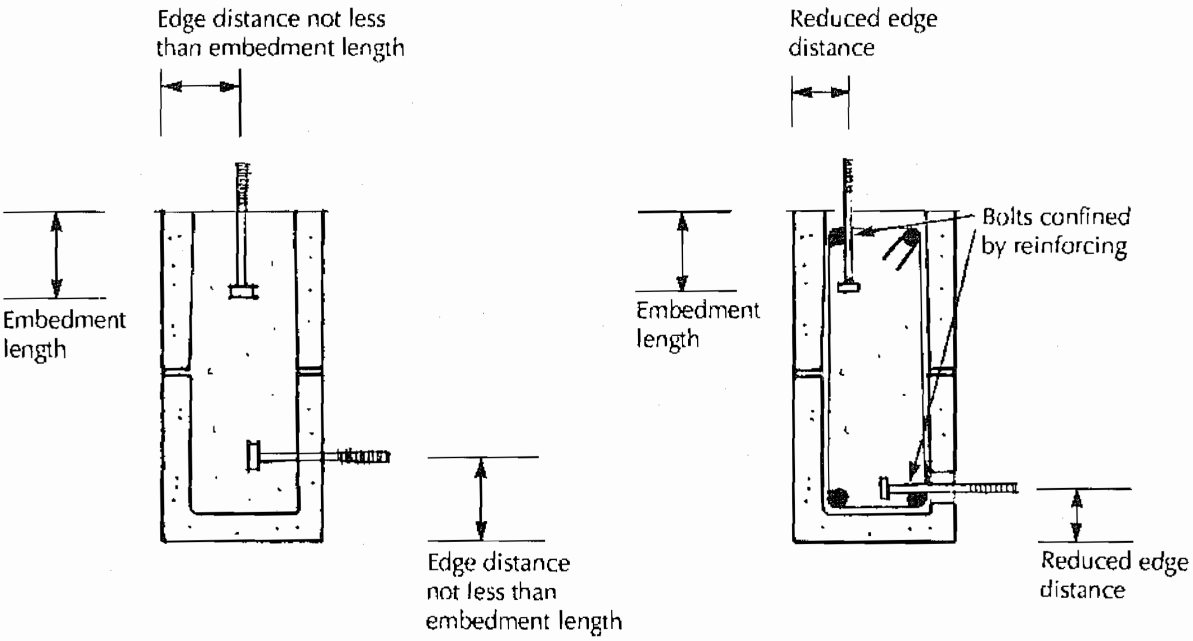
Edge distance not less than embedment length
Embedment length
Edge distance not less than embedment length
Reduced edge distance
Bolts confined by reinforcing
Embedment length
Reduced edge distance

Figure C2 – Edge distance requirements

APPENDIX D
PARTIAL FILL REQUIREMENTS

(Normative)

D1 Scope

This Appendix details the special requirements for partial fill masonry. It is intended as a series of points that modify the application of the principles contained in the sections of this Standard and not as a stand alone section.

D2 Ductility

Buildings containing partial fill as the main structural load resisting system shall be limited to a structural ductility factor of $\mu = 2$ unless a special study shows higher ductilities may be used.

D3 Ductile frames

Partial fill shall not be used as the main structural load resisting system in buildings designed as ductile frames.

D4 Confining plates

Partial fill shall not be used in the design of walls requiring confinement plates unless special study shows this to be acceptable.

D5 Axial loads

Area of face-shells (see AS/NZS 4455) in empty cells shall be ignored for axial load calculations.

Area of face-shells in empty cells shall be ignored when calculating buckling loads of walls.

D6 Shear loads

Shear loads are to be allocated to grouted cells only. Face-shells of grouted cells may be included in shear calculations. Face-shells in ungrouted cells may not be included. See figure 10.1.

D7 Flexural loads

Area of face-shells in empty cells shall not be included in the calculation of a depth of equivalent rectangular stress block.

CD7
When calculating in-plane flexural capacity of partially grouted walls, it is important to check that the neutral axis depth does not become so large that it moves into an un-grouted cavity. This will cause the face-shells of the masonry unit to become overloaded and localized spalling will occur. A solution is to grout the cells on the end of a wall panel to ensure that the neutral axis is located in a fully grouted section of the wall.

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D9 Minimum reinforcement

Minimum reinforcement shall follow the rules for solid filled masonry, but the actual grouted area of the wall shall be used, not the gross cross-sectional area.

D10 Horizontal reinforcement

All partially filled walls shall contain, as a minimum, a horizontal bond beam of minimum dimensions 140 mm wide by 190 mm deep reinforced with at least two D16 bars with R6 links at 600 mm centres at each floor level.

CD10
Bond beams may also be constructed 390 mm deep.

APPENDIX E
SPECIFIC DESIGN FOR SMALL REINFORCED MASONRY BUILDINGS

(Normative)

E1 Scope

This normative Appendix specifies the design methods to be used for small reinforced masonry buildings.

The dimension limitations for these buildings shall comply with clause 1.13 of NZS 4229 except as follows:

(a) The total height from covered ground level to highest point up to roof shall not exceed 12 m;

(b) The plan footprint area shall not exceed:

  (i) 720 m^2 for single storey masonry buildings;

  (ii) 300 m^2 for two storey masonry buildings;

  (iii) 400 m^2 for two storey masonry buildings where the upper storey is constructed of timber and the external wall of the lower storey is of masonry supported on a concrete slab-on-ground, concrete or masonry footings or masonry walls;

  (iv) 300 m^2 for two storey buildings and two storey with attic buildings constructed with upper storey or storeys of timber supported on a lower storey of masonry with the top storey contained within a roof space.

Amd 1 Dec '06 The imposed action shall be determined from AS/NZS 1170.

CE1
NZS 4229:1999 Concrete Masonry Buildings Not Requiring Specific Engineering Design primarily covers two storey residential buildings. The purpose of this Appendix is to permit the use of specific design by extension of the design philosophies and live loading used to generate the tabular design in NZS 4229.

E2 Material and strength properties

The material and strength properties of 3.4 shall apply.

CE2
The basic design parameters used to generate the tables in NZS 4229 used a f'_m value of 8.0 MPa and a maximum shear of 0.24 MPa, corresponding to the values previously specified in NZS 4230. The design parameters for specific design of small reinforced masonry buildings can include the material provisions of this Standard, i.e. 12.0 MPa.

E3 Bracing capacity

Specific design for small reinforced masonry buildings shall follow the provisions of A3.2.2 of NZS 4229 but the strength reduction factors in this Standard may be used in lieu of those given in table A3.1 of NZS 4229. Amd 1 Dec '06

CE3
The design provisions of NZS 4229 are further detailed in References E1 and E2 for in-plane behaviour, with supporting experimental data also presented in Reference E3.

E4 Bond beams

The design of bond beams shall follow the rationale of A3.3.1 of NZS 4229.

CE4

A series of bond beam tests and computer modelling was used to establish a revised ultimate strength value. See References E4 to E8.

The ultimate limit state performance of the bond beams tested was approximately 3 times that predicted by conventional section analysis. Tables 8.3 and 10.1 of NZS 4229 were developed principally from the test results by using a factor of 2 on conventional section analysis with adjustments for horizontal loading conditions arising from masonry veneer for example and in two storey construction from the intermediate floor. Such development of the values in the tables is only permissible within the limited building parameters set by this Standard. Such span values should not be applied to other masonry structures outside the jurisdiction of the Standard without specific engineering design considering the applicability of the test data to the particular project.

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E2 Ingham, J.M., Davidson, B.J., Brammer, D.R., and Voon, K.C. "Testing and Codification of Partially Grout-filled Nominally-reinforced Concrete Masonry Subjected to In-plane Seismic Loads", Journal of the Masonry Society, Vol. 19, No. 1, 2001, pp. 83-96.

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E7 Kelly, T.E., Holmes Consulting Group, "Face Load Testing of Masonry Walls, Supplementary Analysis of Wall with Central Opening", June 1997, Report prepared for the Cement & Concrete Association of NZ.

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APPENDIX F
MASONRY VENEERS

(Informative)

F1 General design principles

F1.1 Specific design of veneers

Amd 1 Dec '06

Reinforced and unreinforced masonry veneers, their wall ties and supporting structure, should be specifically designed to cater for in-service loads and ensure there is adequate strength and compatibility between the component parts of the masonry veneer system.

F1.2 Influence of properties on design

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The seismic performance of a masonry veneer tied to a structure is influenced by the stiffness, strength and ductility of the component parts of the veneer, its wall ties and the supporting structure[F1]. These properties should be considered in the design of a masonry veneer vulnerable to earthquake actions, both in the plane of the veneer and at right angles to it.

F1.3 Seismic response

Masonry veneers and their fixings should not adversely modify the designed seismic response of the structure.

F1.4 Minimum damage to veneer

Every masonry veneer should be designed and detailed to ensure that it suffers a minimum of damage during deformations of the supporting structure under seismic conditions.

F1.5 Differential movement

Masonry veneers should be designed and detailed to accommodate long-term in-service differential movement between the building structure and the veneer cladding without either damaging the masonry veneer or structure or endangering the face load capacity of the wall ties or their connections[F2, F3].

F1.6 Gravity loads

Masonry veneers should not support gravity loads from other parts of the structure, and the supporting structure should be strong enough to support gravity loads from the masonry veneer[F4, F5].

F1.7 Isolated masonry veneers

Masonry veneers which are structurally isolated from the seismic force resisting system should be designed as elastic responding structures in the plane of the veneer and should be detailed to have both proportions and shape to resist any in-plane seismic-induced forces generated through their own mass.

F1.8 Support of unreinforced masonry veneers

Unreinforced masonry veneers should be tied at regular centres to supporting walls and should conform with the dimensions and support requirements in F2 and F3.

F1.9 Regular support of reinforced masonry veneers

Unreinforced masonry veneers should be tied at regular centres to supporting walls, except as covered by F1.10, and should comply with the dimensional and support requirements of F2 to F4 inclusive.

F1.10 Irregular support of reinforced masonry veneers

Reinforced masonry veneers not tied at regular centres to supporting walls should comply with the dimensional and support requirements of F2 to F4 inclusive.

F1.11 Stiff wall ties

Structures braced by stiff concrete or masonry shear walls may have masonry veneers fixed with stiff wall ties complying with the provisions of AS/NZS 2699.1 provided it can be shown that the veneers will not materially alter the seismic response of the structure, or their wall ties be overstressed[F6].

F1.12 Flexible wall ties

Masonry veneers supported by walls, where the seismic resisting structure consists of a flexible frame, sheet bracing on timber framing or timber or light steel bracing system, should be structurally separated by flexible wall ties to the provisions of AS/NZS 2699: Part 1.

F1.13 Shear strength of masonry veneer

The evaluation of shear strength of a veneer should be in accordance with section 10. The total width of unreinforced veneer panels should be considered in determining their shear strength[F7, F8, F9].

F1.14 Flexural strength of masonry veneer

The evaluation of flexural strength of reinforced masonry veneers shall be determined from the masonry to mortar bond strength of the proposed construction, as tested in accordance with NZS 4210. In the construction documents the designer shall specify the minimum masonry to mortar bond strength used in the design in the construction documents[F10].

F2 Dimensional limitations

F2.1 Minimum thickness of regularly supported masonry veneers

Unreinforced masonry veneers and reinforced masonry veneers tied at regular centres to a supporting wall, and not grouted to the supporting wall, shall have a minimum thickness of 70 mm.

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F2.2 Minimum thickness of irregularly supported reinforced masonry veneers

Reinforced masonry veneers tied at irregular centres to a supporting wall and not grouted to the supporting wall should have a minimum thickness of 90 mm. Each reinforced cell should have a clear flue area of not less than 2000 mm^2 and a clear flue dimension of not less than 32 mm.

F2.3 Thickness of unsupported reinforced masonry veneers

Reinforced masonry veneers not tied to a supporting wall should comply with the thickness requirements of section 7.

F2.4 Maximum height of unreinforced masonry veneers

Unreinforced masonry veneers should not exceed the heights in table F1.

F2.5 Vertical support of unreinforced masonry veneers

Unreinforced masonry veneers should be supported vertically on a foundation and at such floor levels that the cavity does not extend more than two storeys in height.

F2.6 Vertical support of reinforced masonry veneers

Reinforced masonry veneers should be supported vertically at the foundation and at such floor levels that the cavity does not extend more than 11.0 m in height.

Table F1 – Maximum height of unreinforced masonry veneers which are subject to specific design (m)

Construction of support walls	Timber	Concrete or masonry
At general locations (Note 1)	7.5	9
Adjacent to egressways and public places (Note 2)	3	3

NOTE –

(1) The height is measured from the top of the foundation or foundation wall to the top of the wall except that where the top of the wall is sloping the maximum height is measured to the higher of:

- (a) The mid-height of the sloping portion; or
- (b) To that level above which the area of masonry veneer wall, inclusive of openings does not exceed 2.0 m^2.

(2) The height is measured from the ground surface on which people will travel to the top of the wall. This condition applies to all areas where there would be a danger from falling masonry to people leaving the building in an emergency situation. It does not apply to single unit dwellings or where there is a protective canopy.

F3 Wall ties

F3.1 Wall tie design requirements

Wall ties should comply with the requirements of NZS 4210 and AS/NZS 2699:Part 1, and should be capable of accepting differential deflection, including seismic displacement, and imposed forces between the masonry veneer and the supporting structure in the plane of veneer.

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F3.2 Forces in wall ties

Forces in wall ties to resist veneer face-loads in accordance with AS/NZS 1170 should be determined by rational analysis, taking into account the relative stiffnesses of the masonry veneers, wall ties and their support as a composite structure[F4, F5]. The maximum *E* value of veneers, for the purpose of determining out-of-plane stiffness, may be taken as 4 GPa unless otherwise determined by test data[F11].

F3.3 Spacing of ties in unreinforced masonry veneers

In unreinforced masonry veneers the spacing of ties should conform with NZS 4210, except that in gable areas the spacing should be decreased to one half the standard spacing for a raking band width of 800 mm following the top of the veneer.

F3.4 Spacing of ties in regularly supported reinforced masonry veneers

In reinforced masonry veneers tied at regular centres to a supporting wall, ties should be built into reinforced cells where practicable, and the distance between the ties may be increased so that a maximum of 0.48 m^2 of face area of veneer is associated with each tie.

F3.5 Spacing of ties in irregular supported reinforced masonry veneers

In reinforced masonry veneers not tied at regular centres to a supporting wall, ties should be built into reinforced cells and should coincide with each vertical reinforcing rod.

F3.6 Openings

Wall ties around openings should comply with NZS 4210.

F4 Reinforcement

F4.1 Protection of exposed reinforcement

Reinforcement should be galvanized or otherwise protected where the cover requirements are not satisfied.

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